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HIGH WALLS OR DAMS TO RESIST THE PRESSURE OF WATER.

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WITH DISCUSSION.

In recent investigations, principally by French engineers, the crushing strength of the material of which the wall is constructed is made a controlling factor. No objection could be made to this if the numerical values of the resistance to crushing were determined under the same conditions as in the cases to which they are applied. The mortar is usually weaker than the stone. If made of Portland cement and thoroughly set, its resistance to crushing, as deduced from hand specimens, may be taken at a ton per square inch. Experienced engineers adopt one-tenth of the breaking weight as the safe load in ordinary masonry, to be reduced to a fifteenth or twentieth in rubble masonry.* One-tenth of a ton per square inch is equal to 200 pounds per square inch, and

* Poncelet, Traite de Mécanique Industrielle, 2d Edition, Vol. I, Sect. 264.

this is about the maximum pressure to which the masonry is subjected in any of the high reservoir walls or dams that have been constructed, so far as we have accounts of them.

Are we justified in applying the results of experiments on the resistance to crushing made on small specimens to large masses of the same material? I think not. Two-inch cubes of the strongest granite are crushed with a load of about 12 tons per square inch, equivalent to a column of the same material 1 inch square and nearly 4 miles high. If the results of the experiments on the 2-inch cubes would apply, a granite formation would be crushed at this depth; and granite being the strongest of rocks, at this and all greater depths, the materials that are solid near the surface of the earth must all be crushed. In the absence of proof that this is the condition, in the light of experience it will be admitted that it is not so: but, on the contrary, we should expect to find granite and all other rocks at that depth to be at least as sound as at any less depth.

What are the differences in the conditions in the two cases?

Any part of the interior of a great mass of uniform material, and subject to uniform pressure, is prevented from yielding laterally by the adjoining parts, which are under the same pressure, and react with equal force; while in small masses the lateral support is slight. The material in the interior of the great mass is compressed into a somewhat smaller volume than it would be under less pressure, but within its original horizontal dimensions, and is not fractured, as that would require a lateral movement of some of its parts, which cannot take place.

Somewhat similar effects have been observed in hand specimens of different sizes subjected to crushing pressures. We are indebted to General Gillmore* for valuable experiments on this and other points. He found that the resistance to crushing per square inch of surface increased rapidly with the area of the bed subjected to the pressure. For instance, he found that a 1-inch cube of yellowish-gray Berea sandstone was crushed with a pressure of 7 000 pounds. Four of these placed side by side would, therefore, require 28 000 pounds to crush them; while a block 2 inches square and 1-inch thick, having, of course, the same area of bed as the four 1-inch cubes, had a compressive strength of 38 000

* Official report to the Chief of Engineers of the United States Army, on the compressive strength, specific gravity and ratio of absorption of the building stones in the United States, by Q. A. Gillmore, Lieutenant-Colonel of Engineers, Brevet Major-General, U. S. A., dated August 10th, 1875.

pounds, or about thirty-six per cent. greater than the sum of the compressive strength of the four 1-inch cubes. In the same report he gives, in detail, the results of experiments showing the increase in resistance to crushing per square inch of cubes of two varieties of Berea sandstone as the size of the cube is increased. He found that his average results are given very closely by the formula $y = C\sqrt[3]{x}$, in which $y =$ the average compressive strength in pounds per square inch of surface, and $x =$ the side of the cube in inches. The following tables, Nos. 1 and 2, contain summaries of the results given in General Gillmore's Tables, Nos. 1, 1a, 2, 2a, to which I have added in the final columns, approximate estimates of the resistance to crushing at the middle of the surfaces, which I arrive at as follows:

In the comparison of the compressive strength of a 1-inch cube and a block 2 inches square, the thickness was the same, and the increased strength was evidently due to the increased area. The parts near the sides of the cube and block must have been of substantially the same strength per square inch, and the increased strength of the block must have been mainly in its central parts, and the maximum strength near the middle. Applying the same reasoning to the experiments on the sandstone cubes in Tables Nos. 1 and 2, I assume that the resistance per square inch of surface at the edges of the cube is very small, and at the middle it is a maximum. If the resistance per square inch of surface to crushing increases uniformly from the edge of the cube to its middle point, the total pressure on the surface may be represented by a pyramid. Let *A B C D*, Fig. 1, Plate XVIII, be a plan of the cube surmounted by the pyramid, and *E F G* be a vertical section of the pyramid through *H I* and *K L*, and let *E F N M* be a prism whose base is the surface of the cube, and *E M* and *F N* the average resistance per square inch to crushing, then the volume of the prism will represent the total resistance of the cube to crushing; and in order that the volume of the pyramid may have the same volume as the prism and also represent the total resistance of the cube to crushing, its height *O G* must be three times the height *O P* of the prism; that is, the resistance per square inch to crushing at the middle of the cube, where it is a maximum, is three times the average resistance per square inch of the whole surface.

It will, however, be more rational to assume that the law of the increase of resistance per square inch from the edges to the middle is

represented by a curved line. The data furnished by General Gillmore's experiments may be sufficient to determine its precise form, but if so, I have not been able to find it. Some of its characteristics are, however, obvious. It must be symmetrical about the axis $O\ G$, the maximum ordinate being on this axis. The ordinates at E and F, corresponding to the edges of the cube, must be small, and a tangent at the vertex must be perpendicular to the axis. Two curves with these characteristics are given at Fig. 1, Plate XVIII. The curve $E\ Q\ F$ is a parabola, and the curve $E\ R\ F$ is an ellipse. The solid on the square base, of which the parabola is a section through $H\ I$ and $K\ L$, may be called a paraboloid with a square base, and in order that its volume may be the same as that of the prism $E\ F\ N\ M$ its height $O\ Q$ must be double the height of the prism; that is, the maximum resistance per square inch to crushing is twice the average resistance per square inch of the whole surface.

Similarly for the curve $E\ R\ F$ the solid may be called a semi-prolate spheroid with a square base, and in order that its volume may be equal to that of the prism, its height $O\ R$ must be one and a half times the height of the prism; that is, the maximum resistance per square inch to crushing is one and a half times the average resistance per square inch of the whole surface.

Either of these results would be a sufficient approximation for the present purpose, which is simply to show that the resistance to crushing is much greater at the central parts of a cube of homogeneous material than near the sides.

The estimates of the compressive strength at the middle of the cube given in Tables Nos. 1 and 2, are from the paraboloid, which I take as giving medium results, and according to which the strength at that point is twice the average strength of the whole cube.

In Mr. John Grant's paper on the strength of Portland cement, in the Proceedings of the Institution of Civil Engineers, read December 12th, 1866, he gives the results of numerous experiments on the resistance to crushing of blocks of neat Portland cement, $3 \times 3 \times 1\frac{1}{2}$ inches, of different ages, crushed in a hydraulic press, part flat and part on edge. Ten blocks of each age and in each position were tested.

The following is a summary of the results:

Eighty blocks crushed on edge.

Area exposed to pressure, $3 \times 1\frac{1}{2}$ inches $= 4.5$ square inches.

Average age, in water, seven days; in air, six hundred and ninety-nine days.

Average tensile strength per square inch, 303 pounds.

Average compressive strength per square inch, 4 956 pounds.

Compressive strength $\frac{4956}{303} = 16.36$ times the tensile strength.

Seventy blocks crushed flat.

Area exposed to pressure, 3×3 inches $= 9$ square inches.

Average age, in water, seven days; in air, six hundred and sixty-six days.

Average tensile strength per square inch, 327 pounds.

Average compressive strength per square inch, 5 882 pounds.

Compressive strength $\frac{5882}{327} = 17.99$ times the tensile strength.

Comparing the two results, the blocks crushed flat have double the area exposed to pressure and one-half the thickness of the blocks crushed on edge, and have a resistance to compression per square inch about 11 per cent. greater.

In General Gillmore's experiments, the resistance per square inch to crushing cubes of yellowish gray Berea sandstone for the area of 9 square inches is about 12 per cent. greater than for 4.5 square inches area ; the two results indicating nearly the same increase of strength per square inch with the same increase of area exposed to pressure.

TABLE No. 1.

SHOWING the increase in resistance, per square inch, of cubes of yellowish gray Berea sandstone as the size of the cube is increased. These were pressed upon by wooden cushion blocks, ranging from one-sixteenth to a little over three-eighths of an inch in thickness, as the cubes increased, and of the same dimensions as the stone. They were all broken on bed.

Side of cube. Inches.	Number of cubes tested.	Average compressive strength per square inch. Pounds.	Compressive strength computed by the formula, $y=7\,000\sqrt[3]{x}$ Pounds per sq. in.	Estimate of the compressive strength at the middle of the cube. Pounds per sq. in.
0.25	8	4 992	4 410	8 820
0.50	10	6 080	5 558	11 116
0.75	7	6 347	6 356	12 712
1.00	9	6 990	7 000	14 000
1.25	9	7 342	7 539	15 078
1.50	10	8 226	8 015	16 030
1.75	9	9 310	8 435	16 870
2.00	9	8 955	8 820	17 640
2.25	7	9 130	9 170	18 340
2.50	10	8 856	9 499	18 998
2.75	10	9 838	9 807	19 614
3.00	3	10 125	10 095	20 190
4.00	1	11 720	11 112	22 224

TABLE No. 2.

SHOWING the increase in resistance, per square inch, of cubes of blue Berea sandstone as the size of the cube is increased. These were pressed upon by hardened steel surfaces. They were all broken on bed.

Side of cube.	Number of cubes tested.	Average compressive strength per square inch.	Compressive strength computed by the formula, $y=9\,500\sqrt[3]{x}$	Estimate of the compressive strength at the middle of the cube.
Inches.		Pounds.	Pounds per sq. in.	Pounds per sq. in.
1.00	11	9 500	9 500	19 000
1.25	10	10 300	10 231	20 462
1.50	8	10 730	10 877	21 754
1.75	9	11 377	11 447	22 894
2.00	10	12 000	11 970	23 940
2.25	10	12 500	12 445	24 890
2.50	9	10 635	12 895	25 790
2.75	7	13 200	13 309	26 618

In the experiments on crushing the cubes of sandstone and blocks of cement with a hydraulic press, it would seem that an equal pressure is applied to each square inch of the surface, and it evidently would be so, if applied to a perfectly rigid mass. All substances, however, so far as known, are compressible in some degree, stone and cement not being exceptions. Under the conditions in which the pressure is applied in these experiments, the amount of compression must be the same at each square inch of surface, but the resistance to compression is much greater at the central parts than near the edges. In other words, the pressure must be greater to produce the same amount of compression at the central parts than near the edges. The cause of this is, evidently, the less freedom of the central parts to yield laterally to the pressure, and the practical effect is to transfer part of the pressure from the parts near the edges to the central parts.

H. Gollner* reports some experiments made at Prague on the crushing strength of cylinders and cubes of Portland cement. The average

* Experiments on the strength of cements and on the bending of bars, by H. Gollner. Abstracts of papers in Foreign Transactions and Periodicals, in Proceedings of the Institution of Civil Engineers, 1877-78.

crushing strength per square inch of the cubes was about seventy-two per cent. of that of the cylinders. Fig. 2, Plate XVIII, represents the surface of the two blocks to which the pressure is applied. Taking the area of the inscribed circle as the unit, the area of the square is 1.273, and the total pressure upon it is $1.273 \times 0.72 = 0.917$, the total pressure on the cylinder being 1.000. This indicates that the angles of the cube, projecting beyond the cylinder, added nothing to the strength of the block, and that practically the pressure applied to them was transferred to the inscribed circle.

The preceding experimental results, and the deductions therefrom, all relate to hand specimens of stone and hardened cement, of regular prismatic forms, subjected to external pressures; but the deductions therefrom, I believe, may be extended to great masses of any form, subjected to gravity or external force, or a combination of the two; the fundamental principle being, that different parts of the mass yield unequally to pressure, depending on the opportunities to yield laterally to the direction of the pressure, the effect being to transfer pressure from the parts yielding more easily to compression to those yielding less easily.

The transfer of pressure takes place in tunnelling through rock, chalk and clay, in cases where the excavation does not require timbering to support the roof before the lining of masonry is completed.

The tunnels under Lake Michigan, at Chicago, were constructed for the purpose of drawing water from the lake for the supply of the city, at a sufficient distance from the shore to be uncontaminated by sewage. They each extend about two miles under the lake. The formation is clay, with occasional pockets and beds of sand of small extent, and some bowlders. The tunnels are mostly through clay, in which the roof did not need support before the masonry was put in.

Fig. 3, Plate XVIII, is a cross section through both tunnels, where the depth of water over them was the greatest. They are both substantially at the same level, their centers being about 65 feet below the surface of the lake, and they are about 50 feet apart. The smaller, or old tunnel, is intended to supply fifty millions of gallons of water in twenty-four hours, and was completed in 1867; it is five feet in diameter inside the lining, which is of brick, and 8 inches thick. The larger, or new tunnel, is intended to supply one hundred millions of gallons in

twenty-four hours, and is seven feet in diameter inside the brick lining, which is one foot thick. It was completed in 1874.

The excavation was "generally through stiff blue clay. * * * It very seldom required bracing, when not left to support itself more than thirty-six hours,"* Professor Andrews describes it "as a solid, impermeable clay."† In the new tunnel the progress was usually about 10 feet per day. The work was carried on by eight-hour shifts. In the first shift the main part of the excavation was taken out; the second shift completed the excavation, trimming it to the exact size and form to receive the lining; the third shift put in the brick lining, which is built up solid to the clay.

The depths of water and clay at the part represented in the section, Fig. 3, Plate XVIII, were about equal. The pressure at the level of the center of the tunnels, before any excavation was made, was

Water	$32.5 \times 62.5 = 2031.25$	pounds.
Clay	$32.5 \times 120. = 3900.00$	"
Total	5931.25	"

or 2.965 tons per square foot of area; and this pressure was, of course, uniform over an area of indefinite extent in all directions. When the excavation was made for the new tunnel, the support for 9 feet in width being taken away, an additional pressure of $9 \times 2.965 = 26.685$ tons per foot in length of the tunnel was transferred to adjacent areas; and not only this, but the pressure on the areas adjoining the excavation on each side was partially transferred to more distant areas, otherwise the clay on the sides of the excavation would be forced in. No yielding of the roof or sides was observed, but it must have taken place, in a slight degree, during the progress of the excavation, in order that the pressure might be transferred. In such cases it is commonly said that an arch is formed in the superincumbent material, which removes the pressure from immediately beneath it and transfers it to the sides. This explanation, of course, involves a small yielding of the material, in order to admit of a new adjustment of the pressures. I think a more rational explanation is, that a beam is formed, carrying a load uniformly distributed and resting on a considerable area on

* First annual report of Department of Public Works of the City of Chicago, for the year ending December 31st, 1876.

† The American *Journal of Science* (Silliman's Journal), Second Series, Vol. XLIII.

each side of the excavation. The beam deflects under the load an amount corresponding to the compression of the material supporting it, and rather more immediately over the excavation. The roof of the excavation is held up by the adhesion of the material to the under side of the beam.

This explanation is represented by the dotted lines in Fig. 3, Plate XVIII, a little above the tunnels; the straight lines *A B*, representing the bottom of the beam before the excavation is made, and the curved line, the bottom of the beam after the excavation is made, the vertical scale being greatly exaggerated. The pressures on the sides of the excavations must be entirely removed, as moist clay in that situation would not carry any sensible load. Similar compressions and transfers of pressure must take place in rock and chalk similarly situated; the magnitude of the movements being inversely as the rigidity of the material.

I propose to apply the preceding deductions to high walls and dams of masonry designed to withstand the pressure of water. I assume that they are constructed of rubble masonry, constituting substantially a homogeneous mass; and I assume, in accordance with preceding experiments and deductions, that it is compressible to the extent of causing the transfer of the part of the pressures, which would otherwise be excessive near the exterior of the section, to the central part of the mass, where the resistance to crushing is much greater. This renders it unnecessary to take into account the crushing effect of the pressures, which we must do if the masonry is assumed to be perfectly rigid.

In determining the section of a wall to resist the pressure of water, I assume that it acts simply by its weight, and that it is built on a rock foundation, and that each foot in length has the necessary stability, independently of its connection with other parts of the wall.

The wall may fail from either of the following causes:

First.—It may be overturned on its base or on some higher joint in the masonry.

Second.—The wall may yield to the pressure by sliding on its base or at some horizontal joint in the masonry.

Third.—The material of which it is composed may be crushed.

As to the first-named cause, the mass of masonry required for stability depends on its weight per cubic foot, and upon the mass and form of the section.

The weight of rubble masonry may be taken as varying from two and one quarter to two and one half times the weight of water, depending on the nature of the materials. The stone being heavier than the mortar, the larger the proportion of stone, the heavier the masonry per cubic foot. If it contains a large proportion of large stone, it may be taken at two and one half times the weight of water, or at $62.5 \times 2.5 = 156.25$ pounds per cubic foot. If mostly of small stone, at $62.5 \times 2.25 = 140.62$ pounds per cubic foot.

On a horizontal base, the form of section having the required resistance to overturning, with the least amount of masonry, is a right-angled triangle. In Fig. 4, Plate XIX, $A\ C\ T$ is such a triangle, the apex A being at the level of the greatest height of water to be provided for. The up-stream side, $A\ C$, opposed to the pressure of the water, is vertical, and I call it the up-stream face, the angle C being the up-stream toe. The hypothenuse $A\ T$, I call the down-stream face, the angle T being the down-stream toe.

In Fig. 4, Plate XIX, let $A\ C\ T$ be the section, in which the resistance to overturning, on the toe T, is just equal to the pressure of the water on the hypothesis that the wall is a perfectly rigid mass, the level of the water on its up-stream side being at the level of the apex A of the triangle.

Put h = the height, $A\ C$.

x = the width of the base, $C\ T$.

w = the weight of a cubic foot of the masonry, that of water being unity.

The weight of the masonry is $\frac{h\,x\,w}{2}$. (1)

Its center of gravity is at g, such that the line $g\ B$ being vertical, $B\ T = \frac{2}{3}$ of $C\ T = \frac{2}{3}x$. Then the moment of the weight of the masonry to overturning on the toe T is

$$\frac{h\,x\,w}{2} \times \frac{2}{3}x = \frac{h\,x^2\,w}{3}. \quad (2)$$

The pressure of the water against the up-stream face $A\ C$, is $\frac{h^2}{2}$, which is to be taken as acting at the center of pressure D, the height above the base, $C\ D$, being $\frac{h}{3}$. Then the moment of the pressure to overturning on the toe T is,

$$\frac{h^2}{2} \times \frac{h}{3} = \frac{h^3}{6}. \tag{3}$$

Equating the moments:

$$\frac{h\,x^2\,w}{3} = \frac{h^3}{6}$$

from which we find:

$$x = h\sqrt{\frac{1}{2\,w}} \tag{4}$$

Taking $w = 2.25$, then $x = 0.4714\ h$.

$w = 2.50$, then $x = 0.4472\ h$.

When $h = 100$ feet, and $w = 2.25$, numerically, the weight of the masonry is $\frac{2.25 \times 62.5 \times \overline{100}^2 \times 0.4714}{2} = 331\,453$ pounds, and the pressure of the water is $\frac{\overline{100}^2 \times 62.5}{2} = 312\,500$ pounds.

In Fig. 4, Plate XIX, g being the center of gravity of the masonry, construct the parallelogram $g\,E\,F\,G$, making $g\,E = 312\,500$ and $g\,G = 331\,453$, and draw the resultant $g\,F$, which intersects the base of the wall at T, where the total pressure falls, and the pressure per square inch is infinite.

Such a wall would have no stability, even if the material had perfect strength and rigidity, which, of course, it could not have.

To give the necessary stability, instead of making the resultant of the weight and pressure intersect the base at the toe T, it is the practice to make the base of such width that the resultant would fall within its middle third. Assuming it to intersect the base at one-third of its width from the down-stream toe, and calling the width x', the weight of the masonry is $= \frac{h\,x'\,w}{2}$. (5)

Fig. 5, Plate XIX, the center of gravity of the masonry being at g, its horizontal distance from the axis of moment is $\frac{x'}{3}$, and the moment of the weight of the masonry is $\frac{h\,x'\,w}{2} \times \frac{x'}{3} = \frac{h\,x'^2\,w}{6}$. (6)

Equating this to the moment of the pressure of water, which is the same as before, we have $\frac{h\,x'^2\,w}{6} = \frac{h^3}{6}$,

from which we find $x' = h\sqrt{\frac{1}{w}}$. (7)

Taking $w = 2.25$, we have $x' = 0.6666\ h$.

Taking $w = 2.50$, we have $x' = 0.6325\ h$.

When $w = 2.25$, the weight of the masonry for a wall 100 feet high, is

$$\frac{100^2 \times 0.6666 \times 2.25 \times 62.5}{2} = 468\ 703 \text{ pounds.}$$

The pressure of water being as before = 312 500 pounds.

In Fig. 5, Plate XIX.

$$C\ T = x' = 0.6666\ h = 66.66 \text{ feet.}$$

$$T\ H = H\ B = B\ C = \frac{x'}{3}.$$

In the parallelogram $g\ E\ F\ G$, make $g\ E = 312\ 500$, and $E\ F = 468\ 703$, and draw the resultant $g\ F$, which intersects the base $C\ T$ at H.

Most ledges of rock have seams in various directions which admit the passage of water. If such is the case with the rock on which the wall we are considering is built, we must assume that the water in the seams is in communication with the water in the reservoir, and that near C, Fig. 6, Plate XIX, is an upward pressure on the masonry equal to the head $A\ C$. The upward pressure may also extend over the whole base of the dam, naturally diminishing towards the toe T, where it would be zero. Supposing it to diminish uniformly from C to T, the average would be equal to that due to half the head $A\ C$, and the total upward pressure would be $\frac{h\ x''}{2}$, x'' being the width of the base $C\ T$. The center of the upward pressure will be at B, one-third of the width of the base from C, and equal to $B\ H$ and $H\ T$. The moment of the same on H is $\frac{h\ x''}{2} \times \frac{x''}{3} = \frac{h\ x''^2}{6}$ (8)

adding this to the moment of the horizontal pressure, which is $\frac{h^3}{6}$, (9)

we have for the sum of the moments of water pressures,

$$\frac{h^3}{6} + \frac{h\ x''^2}{6}. \tag{10}$$

The weight of the masonry is $\frac{h\ x''\ w}{2}$, and its moment on H is

$$\frac{h\ x''\ w}{2} \times \frac{x''}{3} = \frac{h\ x''^2\ w}{6}. \tag{11}$$

Equating the moments, $\frac{h^3}{6} + \frac{h\,x''^2}{6} = \frac{h\,x''^2\,w}{6}$.

From which we deduce $x'' = h\sqrt{\frac{1}{w-1}}$. (12)

Putting $w = 2.25$, we have $x'' = 0.8944\ h$.

Putting $w = 2.50$, we have $x'' = 0.8165\ h$.

When $w = 2.25$, for a wall 100 feet high, the weight of masonry is,

$$\frac{\overline{100}^2 \times 0.8944 \times 2.25 \times 62.5}{2} = 628\,875 \text{ pounds.}$$

Deduct upward pressure of water

$$\frac{\overline{100}^2 \times 0.8944 \times 62.5}{2} = 279\,500 \text{ pounds.}$$

Effective weight of masonry, 349 375 pounds.

The horizontal pressure of the water is the same as before, viz., 312 500 pounds.

Construct the parallelogram $g\ E\ F\ G$ and draw the resultant $g\ F$, which intersects the base at H, at one-third the width of the base from the toe T.

In computing the required dimensions of the parts of bridges and other structures of wood and iron, to sustain permanently given strains, factors of safety are introduced, usually of not less than four; that is to say, the computed strength is four or more times the assumed breaking strain. This is needed, not only to give stability from having a surplus of strength, but in order to provide for the uncertainty as to the original strength of the materials, and also for the depreciation in its strength, which ordinarily takes place from age and long-continued use. In a wall, to sustain the pressure of water, we can and ought to assume a height of water and pressure that cannot be exceeded. The wall, which is assumed to act simply by gravity, has no uncertainty as to its weight, and is unchangeable in its power of resistance, and a much less factor of safety is accordingly needed than in constructions of iron and wood.

The section in Fig. 4, Plate XIX, computed by formula (4) has no factor of safety. The sections in Figs. 5 and 6, Plate XIX, computed by formulas (7) and (12) have factors of safety of two; that is, the weight of the masonry is double that required to resist overturning on the toe T, from the pressure of the water. In the parallelograms of forces in Figs. 5 and 6, $g\ G$ and $E\ F$ represent the weight of the

masonry; if this is taken at one-half its real value, the corresponding parallelograms will be $g\ E\ i\ k$, and the resultants will pass through the toe T, as it does in Fig. 4.

Generally the width of base, when there is no upward pressure of water, for any factor of safety, f, is given by the formula

$$x' = h\sqrt{\frac{f}{2\ w}} \tag{13}$$

and when there is an upward pressure, as taken above,

$$x'' = h\sqrt{\frac{f}{2\ (w-1)}}. \tag{14}$$

As to the second-named possible cause of failure of the wall, viz., that it may yield to the pressure by sliding on its base, or on some horizontal joint in the masonry.

In the section of wall represented by Fig. 5, Plate XIX, the weight of masonry above the base or any joint, h feet below the apex of the triangle, is $\frac{62.5\ h\ x'\ w}{2}$; by formula (7), $x' = h\sqrt{\frac{1}{w}}$; substituting the value of x', we have for the weight, $\frac{62.5\ h^2\sqrt{w}}{2}$.

The pressure of water against the wall for the same height h, is $\frac{62.5\ h^2}{2}$; consequently the weight of the masonry is equal to the pressure against it, multiplied by $\sqrt{w}$.

Taking $w = 2.25$, $\sqrt{w} = 1.5$; that is, the weight of the wall is 1.5 times the pressure against it, and in order to slide, the co-efficient of friction must be less than $\frac{1}{1.5}$, or than 67 per cent.

The experiments of Morin indicate that the co-efficient of friction for Oolitic limestone upon Oolitic limestone, the surfaces in contact being smooth planes, is 55 to 75 per cent. Weisbach gives for the "co-efficient of friction of rest," for stone or brick upon stone or brick, well polished, 67 to 75 per cent.

The wall we are considering is founded upon a surface of rock, which, if not naturally uneven, should be made so. The wall is of rubble masonry, laid in hydraulic mortar, but not in courses. There can therefore be no smooth planes without cutting through parts of the rock or some of the rubble stones. Omitting the cohesion of the mortar, the co-efficient of friction must be many times the superincumbent weight,

instead of two-thirds or three-quarters, as required in order to slide. We may therefore conclude that there is no danger of failure from this cause.

As to the third cause from which the wall may fall, viz., the material of which it is composed may be crushed.

According to the method given by Debauve,* when the reservoir is full, and the resultant of the pressure of the water and of the weight of the masonry intersects the base at one-third of its width from the down-stream toe, the maximum pressure per square inch on the base is at the down-stream toe, and is double what the pressure per square inch would be if the weight was uniformly distributed over the whole width of the base.

When the reservoir is empty the conditions are reversed; the maximum pressure being at the up-steam toe, and equal to double the average pressure on the base of the wall.

The weight of the masonry for any height of wall h, by formula (5) is $\frac{h\,x'\,w}{2}$. Then the average pressure per square inch on the base for $w = 2.25$ is $\frac{62.5 \times 2.25\,h}{2 \times 144} = 0.4883\,h$, pounds,

and the maximum pressure, per square inch, according to Debauve, is

$$2 \times 0.4883\,h = 0.9766\,h, \text{ pounds.}$$

The pressure and the width of the base by formula (7), for several heights of wall, are given in the following table:

TABLE No. 3.

Height of Wall. h Feet.	Pressure, in pounds per square inch, on base.		Width of base x' for $w = 2.25$. Feet.
	Average.	Max. according to Debauve's method.	
100	48.83	97.66	66.66
200	97.66	195.32	133.32
300	146.49	292.98	199.98
400	195.32	390.64	266.64
500	244.15	488.30	333.30

* "Dams and Reservoirs," translated by E. S. Gould from *Manuel de L'Ingenieur des Ponts et Chausees*, by A. Debauve, New York, 1881.

In Fig. 7, Plate XX, the section $A\ C\ T$ is drawn according to formula (7). $T\ S$ represents the pressure at the down-stream toe T, when the reservoir is full; according to the method given by Debauve, the pressure at the up-stream toe C being zero. The area of the triangle $C\ T\ S$ represents the total pressure on the base of the wall, the line $C\ S$ representing a uniform decrease of pressure on the base from T to C.

According to the deductions from the experiments cited above, the pressure is mainly transferred from T to the interior of the base, where the resistance to compression is much greater. The pressures at different points may be represented by the ordinates of the curve $C\ V\ T$, the area between it and the line $C\ T$ being equal to the area of the triangle $C\ T\ S$. I am unable to give the exact form of the curve, but some of its characteristics, to accord with what has been stated, are obvious. The ordinates at C and T must be small; the maximum ordinate must be less than $T\ S$, and be either within or near the middle third of the base. The curve $C\ V\ T$ has these characteristics, and is composed of quadrants $C\ V$ and $V\ T$ of two ellipses, the common ordinate $H\ V$ at the intersection of the resultant $g\ F$ with the base $C\ T$ being 0.6366, $T\ S$ being unity.

The sum of the areas of the two quadrants of ellipses is equal to the area of the triangle $C\ T\ S$, and consequently, represents the total pressure on the base of the wall.

By Table No. 3, in a wall 400 feet high, according to Debauve's method, the maximum pressure is at the down-stream toe T, where it is 390.64 pounds per square inch. If the pressure is distributed according to the curve $C\ V\ T$, the maximum pressure is at V, 88.88 feet from the down-stream toe, where it is $390.64 \times 0.6366 = 248.68$ pounds per square inch.

It will be understood that the curve $C\ V\ T$ is given only as an illustration of the distribution of pressures according to the theory advanced, and with no pretension to precision, either as to the distribution or the determination of the point where the pressure is a maximum. It would be desirable to be able to do so, but it is not essential to the present purpose; the main point in reference to the safety of the wall being that the vicinities of the toes are relieved from dangerous pressures by the transfer of a large part of the same to parts of the base capable of sustaining them without danger of crushing.

We have no means of estimating the amount of compression in different parts of the mass. It may be thought to be proportional to the pressure, but this is compensated for, more or less, by the increased strength to resist compression, depending on the increased lateral support to given areas as we recede from the exterior; the result tending to approach uniformity in the amount of compression at different parts of the mass.

In walls and dams to resist the pressure of water, the width of the base and the pressure on it are proportional to the height, and the part of the base capable of sustaining great pressure without crushing is substantially in the same proportion. If the theory of the transfer of pressure that I have attempted to establish is admitted, the conclusion that there is no danger of failure from crushing in any case must also be admitted.

In the case of the rock on which the wall is built, having seams containing water in communication with the water in the reservoir, by constructing drains sufficiently near to each other in the base of the dam to discharge the water from the seams through the outer toe of the wall, no upward pressure could be communicated to the wall; it would, however, allow some waste of water, and might be objected to as interfering with the homogeneity of the masonry. If drains are not provided, it may be questioned whether the upward pressure would extend beyond the parts of the masonry immediately in contact with the seams.

Assuming that the work is as well done as it can be, that is, that all the stones are perfectly bedded in the mortar, and that all the joints are solidly filled with mortar or concrete, would the upward pressure from the seams be communicated to the entire base of the wall, as is assumed, in the method of arriving at the required width of the base of the wall, by formula (12)? This involves the question as to whether the pressure of water can be communicated through the capillary channels in the mortar. Not finding anything definite on record on this point, I made the following experiments:

Fig. 8, Plate XX, is a section of the apparatus. The cast-iron case, *A A*, is an ordinary reducer for the purpose of connecting an 8-inch pipe with a 6-inch pipe. To fit it for this purpose, heads, *B B'*, were applied, packed with rubber, and kept in place by four bolts, two of which are shown at *C C*. An inner pipe *D*, 3 inches interior di-

ameter, is bolted to the head B', the joint being packed with rubber. The interior of the case A and the pipe D are filled, as represented, with Portland cement mortar, in the proportion of cement one, sharp pit sand two, the brand being, The Brick, Lime and Cement Company, Limited, Durham Works; trade-mark, Murston Works. The mortar was tempered to the usual consistency for use, and was carefully worked into place by stirring it with a rod. Spaces E E and F, to receive the water passing through the mortar, were left at the bottom, which were filled with gravel, and covered with cotton cloth, to prevent the mortar from filling the spaces in the gravel. A space, G, was left between the top of the mortar and the upper head B. This space was connected with the system of pipes laid for supplying water for extinguishing fires in the Lowell factories, by the pipe K, and the spaces E E and F were furnished with discharge pipes I and H, the outlets of which were furnished with stop-cocks or caps. Each of the pipes K H and I was furnished with a pressure gauge L M and N, as represented in the figure.

The interior of the case A was originally protected from oxidation by Dr. Smith's patent varnish. It was feared that this might interfere with the experiment by preventing a perfect contact of the mortar with the inside of the case, and allow the water, under pressure, to follow the joint. To avoid this, the varnish was burnt off.

It is well known that Portland cement mortar expands in setting under water, which would seem to be a sufficient guarantee that the water would not follow the joint; but, as further security, the inner pipe D was put in, and means provided for testing the percolation of water through it, and the amount of pressure transmitted, independently of that transmitted through the mortar between it and the case.

The experiments consisted in noting the pressures at the several gauges at the beginning and end of periods, which gave sufficient time for them to become established. The pressure at the gauge L depended entirely on the pressure in the supply pipe, and was subject to changes of short duration, depending mainly on the drawing of water in a neighboring office. These changes were, however, so infrequent and of such short duration, that they had little influence on the results, and no account has been taken of them. The normal pressure at gauge L was about 77 pounds, and it is to be so taken in all these experiments.

The cement mortar was mixed and put into the apparatus February 28th, 1888, and remained in the air in a cellar, where the average temperature was about 36 degrees Fahr., for seven days. It was then put into a tub of water, in the same cellar, where it remained twenty-one days, the temperature of the water averaging 38.3 degrees. The apparatus was then taken out of the tub, and remained in the air another seven days, the temperature of the air being about 40 degrees. It was then returned to the tub, where it remained another seven days, the temperature of the water being 40 degrees. It was then taken out, and the mortar having become well hardened, the apparatus was completed for the experiments.

Unfortunately, the case *A* was not a sound casting, and there was a minute leak on one side, near *O*, and there were indications of other minute leaks through the head B^1, and at the packing between it and the case *A*. These leakages, undoubtedly, modified materially the results of the experiments on the transmission of pressure.

The packing of the inside pipe *D*, to the lower head *B'*, was tested under the full pressure before commencing the experiments, and was found to be perfectly tight.

The experiments were all made in the cellar, between April 11th and 25th, 1888; the average temperature of the air being about 40.5 degrees.

The pressure gauges were compared, and corrected to gauge *L* as the standard.

Experiments on the Transmission of Pressure through Hardened Cement Mortar.

Experiment 1. From April 11th, 6 o'clock P.M., to April 12th, 9.15 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 65.5 pounds. During the final one hour and twenty minutes there was no change in the pressure at gauge *M*.

Experiment 2. From April 12th, 9.16 A.M., to April 12th, 3.27 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 64.0 pounds. During the final one hour and thirty minutes there was no change in the pressure at gauge *M*.

Experiment 3. From April 12th, 3.28 P.M., to April 13th, 11.16 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from

0 to 62.6 pounds. During the final one hour there was no change of pressure at gauge *M*.

Experiment 4. From April 13th, 11.22 A.M. to April 13th, 5.35 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 59.75 pounds. During the final one hour and twenty-six minutes there was no change of pressure at gauge *M*.

Experiment 5. From April 13th, 6.2 P.M., to April 14th, 8.1 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 57.25 pounds. Period during which pressure at guage *M* was stationary not observed.

Experiment 6. April 14th, 11.42 A.M., to April 16th, 9.14 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went up from 0 to 46.5 pounds. During the final one hour and nine minutes there was no change of pressure at gauge *M*.

Experiment 7. From April 16th, 9.30, to April 19th, 8.12 A.M. Pipes *I* and *H* both closed. Pressure at gauge *M* went up from 46.5 pounds to 72.5 pounds. Gauge *N* not then attached. During the final twenty-one hours and forty-five minutes no change of pressure at gauge *M*.

Experiment 8. From April 19th, 8.12 A.M., to April 20th, 10.45 A.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *M* went down from 72.5 pounds to 39.75 pounds. During the final two hours and forty-two minutes no change of pressure at gauge *M*.

Experiment 9. From April 20th, 10.45 A.M., to April 23d, 11.30 A.M. Pipes *I* and *H* both closed. Pressure at gauge *M* went up from 39.75 pounds to 73.5 pounds. Pressure at gauge *N* went up from 0 to 71.0 pounds. During the final three hours and eighteen minutes no change of pressure at either gauge.

Experiment 10. From April 23d, 11.30 A.M., to April 24th, 3.40 P.M. Pipe *H* open, pipe *I* closed. Pressure at gauge *N* went down from 71.0 pounds to 58.0 pounds. During the final seven hours and thirty minutes no change of pressure at gauge *N*.

EXPERIMENTS ON THE PERCOLATION OF WATER THROUGH THE CEMENT MORTAR IN THE CASE *A*, FIG. 8, PLATE XX. CEMENT 1, SHARP PIT SAND 2, BLOCK 15.84 INCHES THICK, SIX TO EIGHT WEEKS OLD.

Experiment 1. From April 12th, 10.55 A.M. to April 12th, 1.58 P.M. Pipe *H* closed, pipe *I* open. Pressure at gauge *L*, 77.0 pounds. Pressure at gauge *M* went up from 27.0 pounds to 64.0 pounds. Drip from pipe *I* in 183 minutes, 1.453 pounds. Rate of percolation in twenty-four hours,

$$\frac{1440 \times 1.453}{183} = 11.433 \text{ pounds.}$$

Experiment 2. From April 24th, 3.53 P.M., to April 25th, 3.53 P.M. Pipes *I* and *H* both open. Pressure at gauge *L*, 77.0 pounds. Drip from pipe *I* in the twenty-four hours, 8.256 pounds. Drip from pipe *H* in the twenty-four hours, 1.762 pounds. Total amount of percolation in the twenty-four hours, 10.018 pounds.

In the experiments on the transmission of pressure through the mortar, Nos. 1, 2, 3, 4, 5 and 6, there is a gradual diminution in the final pressure at gauge *M*, which indicates the pressure transmitted through the mortar between the water spaces *G* and *F*. During all these experiments the pipe *I* was open, and there was no pressure in the space *E*, which afforded a free outlet for the water percolating through the mortar between the pipe *D* and the case *A*; while in the space *F* there was a considerable pressure. The percolation through the pipe *D* must have ceased as soon as the space *F* was filled with water, if there was no leakage from it, which we have assumed to be the case.

The pressures in the mortar at the top of the pipe at *P*, and in the space *F*, must then have been the same, and equal to the obstruction to the percolation through the mortar between *P* and the space *E*. The course of the flow through the mortar, immediately over the pipe *D*, would be deflected to the space between it and the case, as shown by the arrows. The pressures indicated by the gauge *M* are, therefore, on the assumption that there was no leakage from *F*, the pressures in the mortar at the top of pipe *D*.

Our water supply contains more or less sediment, particularly at the season when these experiments were made; and to avoid interference

with the percolation through the mortar, from its deposition, the water was first passed through a sand filter. On examination of the surface of the mortar after the experiments were made, it was found to be covered with a fine red sediment, evidently, in part, rust from the pipes. This undoubtedly caused the reduction in the rate of percolation from April 12th to 24th, as shown in experiments 1 and 2. This reduction in the percolation, combined with the leakage, accounts for the diminution of pressures in experiments 1 to 6 on the transmission of pressure.

In experiment 9 the only outlet for the percolation was the leakage, and the pressure attained in the space F was within 3.5 pounds of the pressure in the space G, and the pressure in E within 6.0 pounds; and it is reasonable to assume that if there had been no leakage, the pressures would have been the same at all the gauges, except that due to their different elevations, which would give about 1 pound more at gauges M and N than at gauge L.

These experiments are on a small scale with an imperfect apparatus; but I think we are justified in inferring from them that pressure can be transmitted through hardened mortar composed of one part of Portland cement and two parts of sharp pit sand; and that in the base of a wall founded on a rock, having numerous seams, containing water under a head, the flow from which is not carried off by drains, an upward pressure may be transmitted through the mortar to the entire base.

The experiments on percolation indicate that water under pressure penetrates the mortar, and unless the up-stream face of the wall is protected by a waterproof lining, the mass of the masonry would be saturated. What effect saturation would have on the stability of the wall is a question, but it would appear to be an additional reason for draining the base, as it would be difficult to prevent the penetration of water at every point.

The formulas (7) and (12) and the corresponding Figs. 5 and 6, Plate XIX, are intended to be used only as skeletons or diagrams to be used in designing high walls or dams to resist the pressure of water, in which it is desired that the amount of masonry should be a minimum. The triangle $A\ C\ T$ being first drawn, the apex being at the highest flood line, additions can be made as desired to provide for all the contingencies to which the wall may be subjected, and the wants to be provided for; such as carrying the wall above the flood line, and adding to its thickness at the upper part to resist the pressure of ice and the action

of waves. Such additions will all add to the weight of the wall and increase its stability over that given by the diagram, which should be treated as an inscribed figure indicating the smallest admissible amount of masonry.

In dams across rivers, over which water is intended to flow, sometimes in great volume and carrying large masses of ice, a wide departure from the diagram will be required. It may, however, be of use in such cases as indicating the dimensions required to resist the pressure of the water.

It is often the case that in order to reach a suitable rock foundation a considerable part of the height of the wall is below the permanent level of the water in the ground. In this part of the wall the effective weight per cubic foot of the masonry is only its excess above the weight of a cubic foot of water. The diagrams computed as above consequently apply only to the part of the wall above the permanent level of the ground water; the part below may be considered as a foundation, to which a different computation applies.

Fig. 4

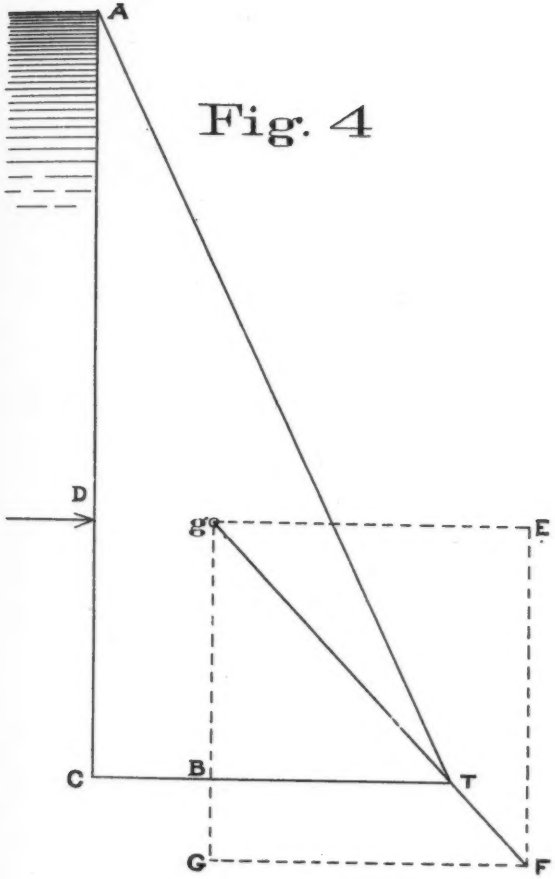


Fig. 5.

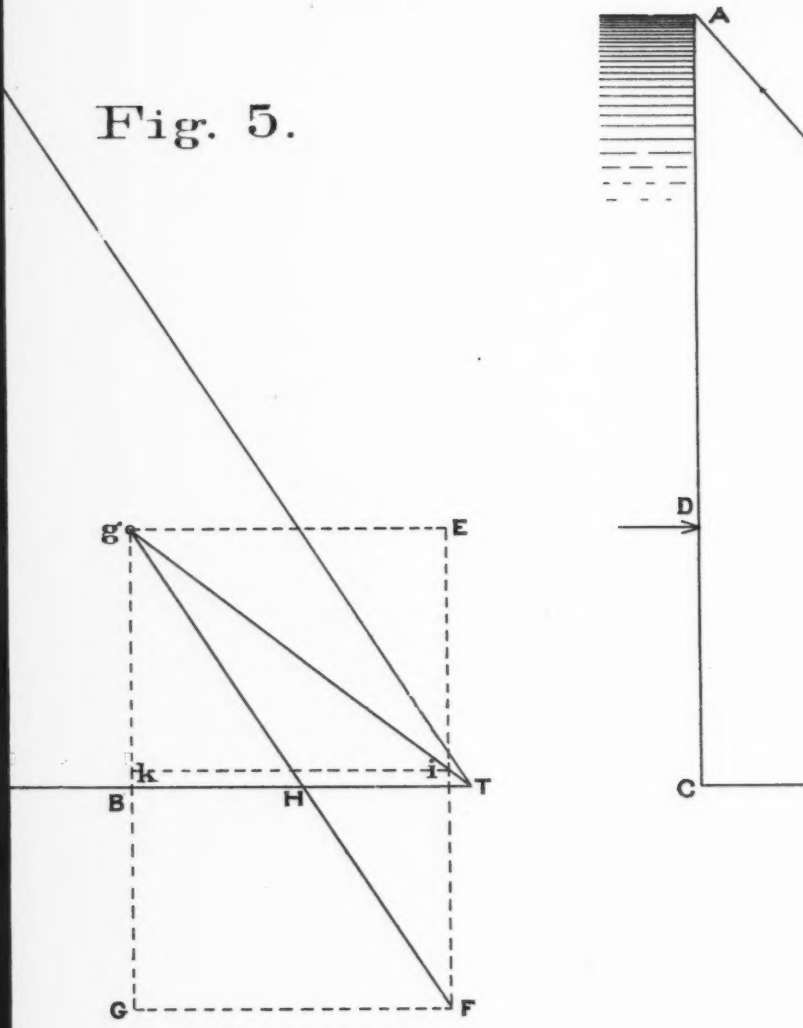
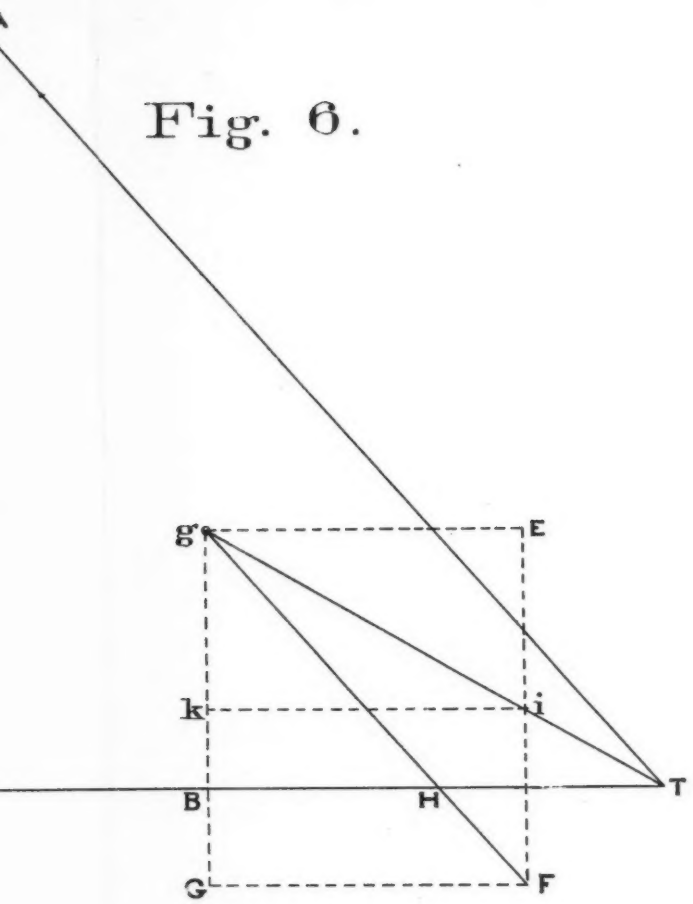


Fig. 6.



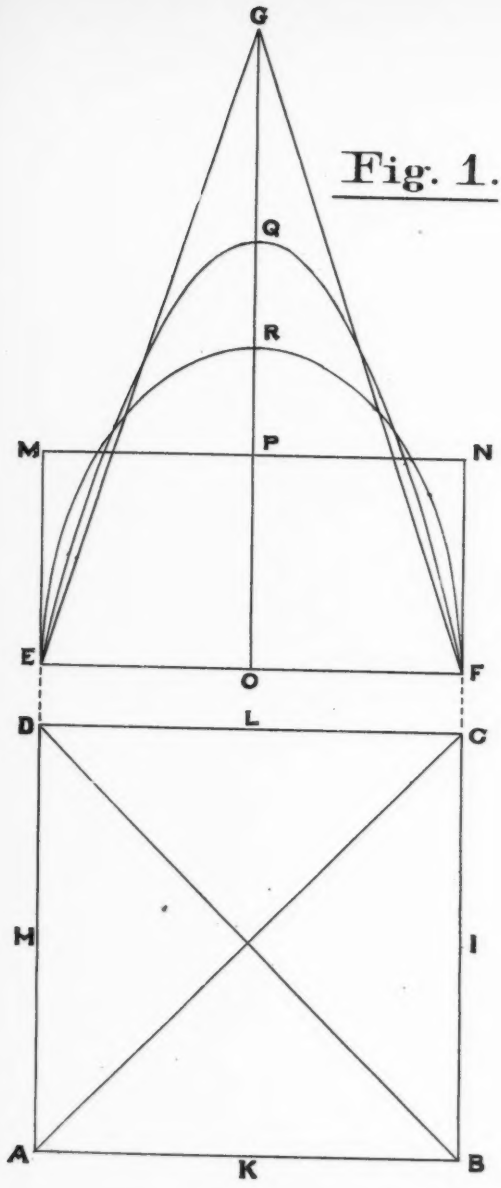
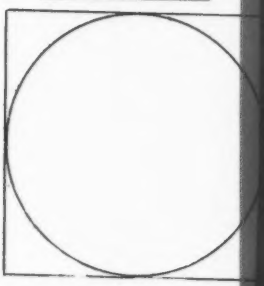
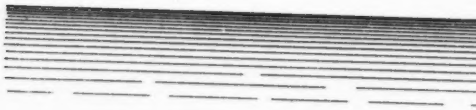


Fig. 1.

Fig. 2.





g. 2.

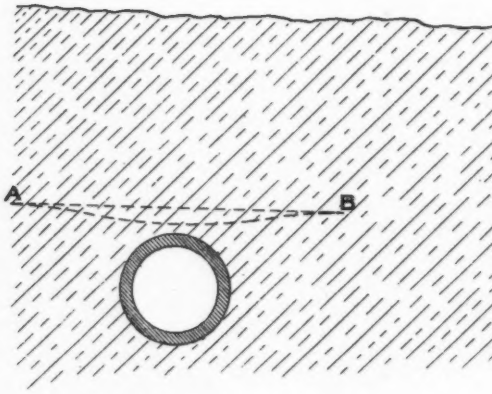


Fig. 3.

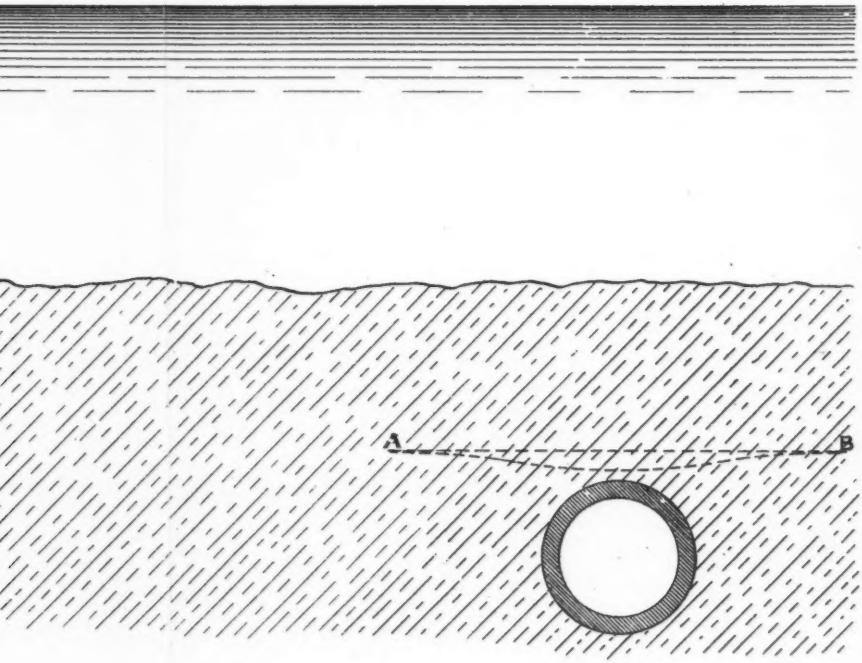


Fig. 7.

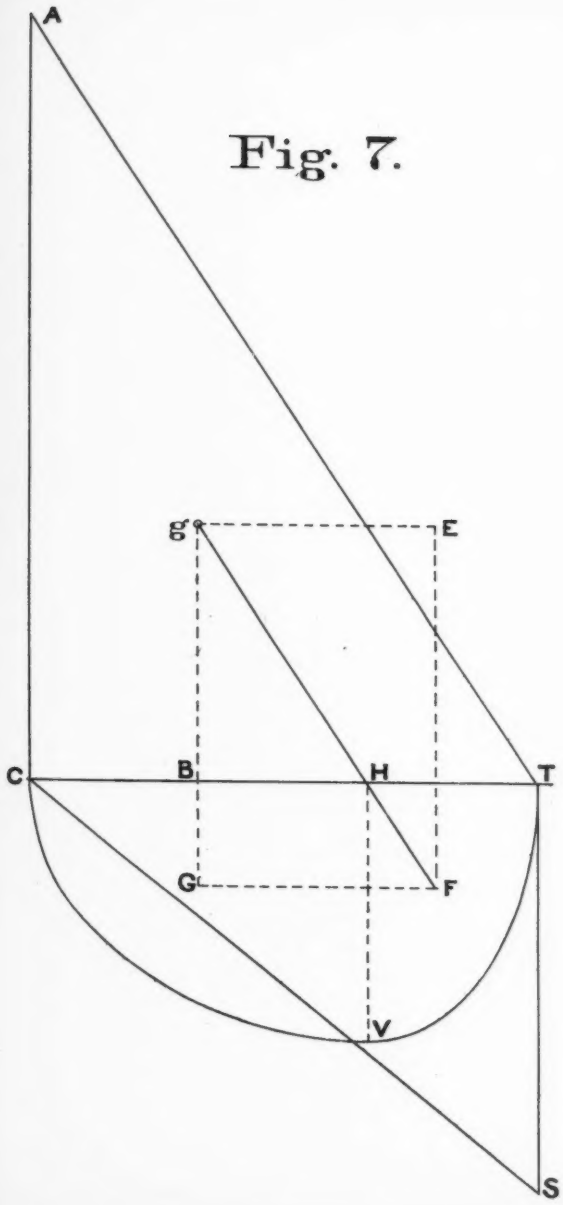


Fig. 8

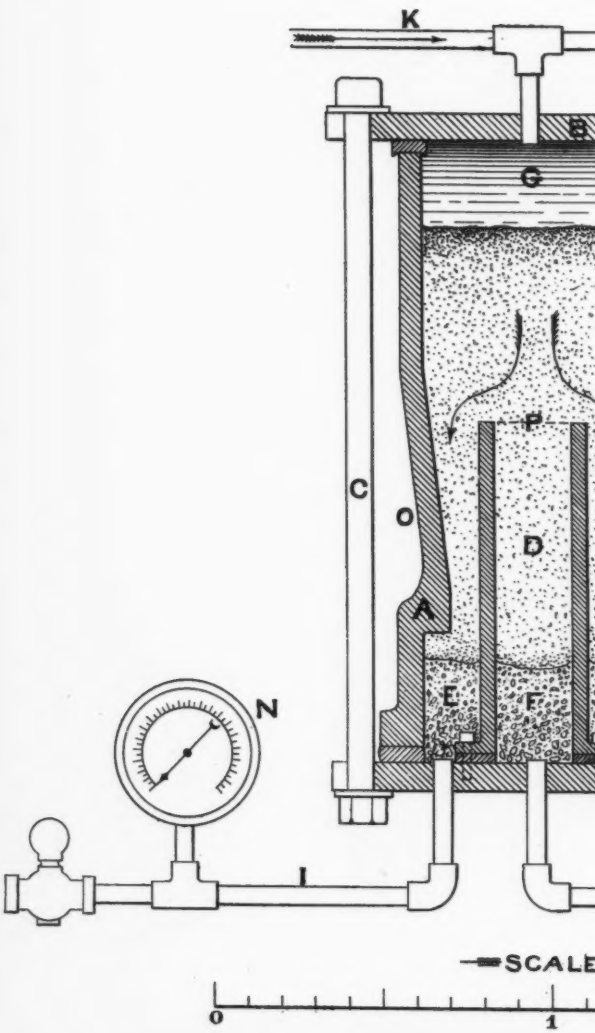


PLATE XX
TRANS. AM. SOC. CIV. ENGRS.
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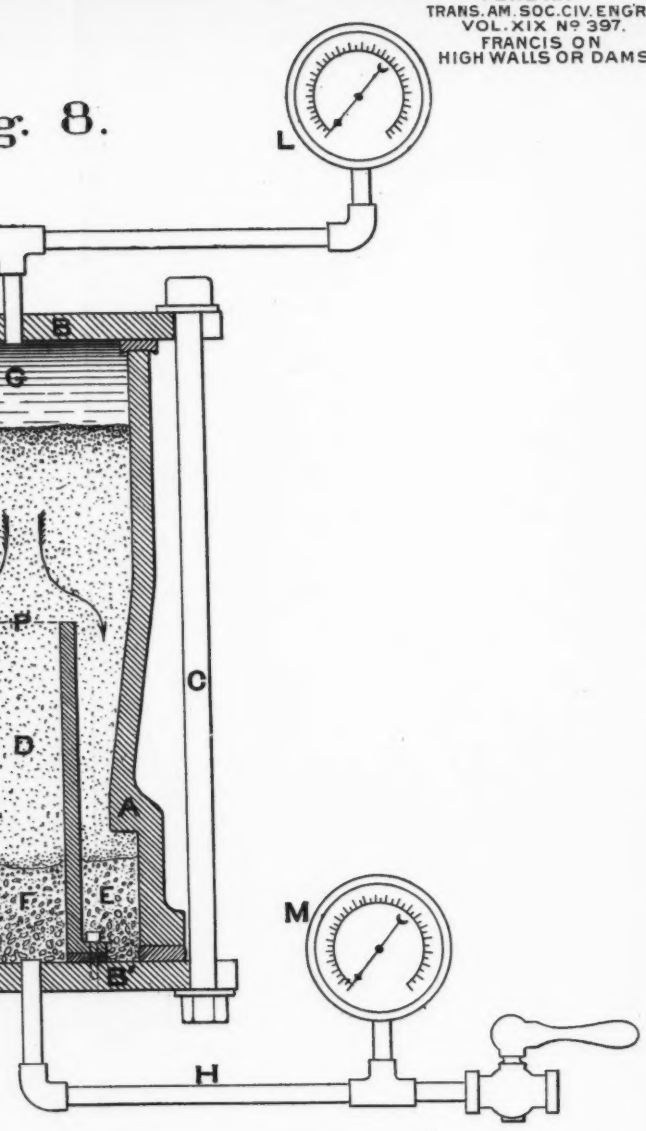
FRANCIS ON
HIGH WALLS OR DAMS

g. 8.



SCALE
1
2 FEET.

DISCUSSION.

EDWARD WEGMANN, Jr., M. Am. Soc. C. E.—The paper of Mr. Francis on high masonry walls or dams, involves the consideration of the following important topics:

First.—The distribution of pressure in a wall of masonry.

Second.—The proper form of profile for a masonry dam.

Third.—The effect of the perviousness of the masonry upon the stability of a dam.

With reference to the first of the above topics, Mr. Francis makes a wide departure from the views hitherto entertained. The usual formulas for the distribution of pressure in a wall have been derived by supposing the masonry to be rigid. Under this assumption, the pressures on the foundation must either be uniformly distributed or form a uniformly varying stress, giving the maximum pressures near the front or back face of the wall. While this hypothesis, which neglects the elasticity of the masonry, is evidently inaccurate, it leads to formulas which are certainly safe, as they exaggerate the pressures in the weakest parts of the wall, namely, near its faces.

In marked opposition to the assumption just stated, Mr. Francis considers that the pressures will be practically zero at the faces, and that they will distribute themselves in accordance with the power of resistance of the various parts of the wall. As a result of this view, he reaches, on page 156 of his paper, the conclusion that it is "unnecessary to take into account the crushing effect of the pressures, which we must do if the masonry is assumed to be perfectly rigid."

This new view of the distribution of pressure is based upon the following facts:

First.—The experiments of General Gillmore have shown that the resistance to crushing of hand specimens increases rapidly with the area of the bed subjected to the pressure—a result which is doubtless due to the additional strength the interior parts of the specimens receive from the lateral support of the outer portions.

Second.—The rock formations of the crust of the earth sustain safely pressures which would crush small specimens.

Third.—When a tunnel is driven through rock or even through the clay bottom of a lake, the pressures of the mass directly over the tunnel are safely transmitted to either side.

With reference to the experiments upon the crushing strength of hand specimens, I must state that they do not appear to me to present a very close analogy to the conditions of pressure in a masonry wall or dam. As these specimens were subjected to a uniformly distributed pressure between two practically rigid plates, it is evident that the outer weaker parts could not be crushed until the inner portions had been

considerably compressed. The experiments would have presented a greater similarity to the ordinary conditions in a course of masonry in a wall or dam if the specimens could have been crushed between two flexible plates, with the pressure varying from a given edge to the opposite one. Under such circumstances, it cannot be doubted that an outer portion of a specimen might have been crushed before the inner parts had been much strained.

To show that such a result might readily occur in a dam, we will consider the triangular reservoir wall, *A C T*, given in Fig. 7 of Mr. Francis' paper. The effect of the water-pressure against this wall is two-fold. It tends:

First.—To revolve it about the front (down-stream) toe *T*.

Second.—To slide it on the foundation or to shear it apart.

The first tendency evidently causes the pressures on the foundation to increase from the up-stream to the down-stream face of the dam. If the water-pressure could be increased until it finally turned the wall over, the whole pressure would be thrown on the front toe *T*, which would, of course, be crushed.

If we wish to represent the distribution of the pressure on the base graphically, the diagram of stress must fulfill the following two conditions:

First.—Its area must represent the total pressure.

Second.—Its center of gravity must lie in the line of action of the resultant pressure, that is, in the vertical line *H V*.

The triangle *C T S*, which represents the distribution of pressure for a rigid dam, satisfies both of the above conditions, and shows what might have been imagined, an increase of pressure towards the front face. The elasticity of the masonry will, undoubtedly, modify this diagram somewhat, curving the line *C S*, but it can never be like the diagram *C V T* given in Fig. 7 of the paper under discussion, as its center of gravity will evidently not be in the line *H V*, and will therefore not satisfy the second given condition. To obtain a rational pressure-diagram, which shall fulfill both of the requirements stated above, we must adhere very closely to the triangle *C T S*. As the moment of the pressures on the front third of the base *H T*, must balance the moments of the pressures on the other two-thirds, in order to keep the center of gravity of the stress diagram in the vertical line *H V*, the pressures on the base must, evidently, increase considerably from the back towards the front of the wall. Just where the maximum intensity of pressure will occur is unknown, but it must be near the front face.

If we now examine the case when the reservoir is empty, we shall arrive at similar pressure-diagrams, only that their positions will be reversed from what they were when the reservoir was full, the maxima pressures being now near the back face. Another way of arriving at the same conclusion is first to imagine the triangle *A C T* to be composed

of columns of masonry placed side by side. As the pressure on any part of the base would depend simply upon the weight of the column resting on it, the pressures would diminish from the back face to the front. Now conceive the columns of masonry to be united. The elasticity of masonry would undoubtedly modify the pressure diagram somewhat, but I do not think very materially, as the center of gravity of the diagram would have to be in the line *B G*.

The facts stated incline me to urge the adherence to the usual formula for the distribution of pressure. A wall having a symmetrical profile may possibly give rise to pressures like those shown in the diagrams for Fig. 1, but as our present knowledge of this subject is very meagre, I think the safer course is to exaggerate the pressures near the faces of a wall rather than to go to the other extreme by assuming them equal to zero.

The manner in which the pressures distribute themselves in a large mass of clay or rock, through which a tunnel is bored, does not seem to me to bear upon the case of a dam subjected to water pressure; for in the former case we have to deal with symmetrical forces which are, in the main, vertical, while in the latter we have to consider inclined forces, which generally necessitate an unequal distribution of pressure. The equilibrium existing between the thrust of the water, the weight of the dam and the reaction of the foundation is the important principle which determines this distribution, and I think the elasticity of the masonry can modify this only to a limited extent. The relative strength of the different parts of the wall cannot exert an important influence in this matter. I can, therefore, readily conceive how a given course of masonry which lies between two other elastic courses, may be crushed near one face before the strength of the inner parts is fairly exerted. I may also state here that when a tunnel is not subjected to symmetrical forces, or is weaker on one side than on the other, there will always be a strong likelihood of its caving in.

Let us now consider the second topic touched upon in Mr. Francis' paper, viz.: The correct profile for a masonry dam. The early writers upon this subject made their designs conform to the following two conditions:

First.—The pressures in the masonry or on the foundation must not exceed certain safe limits, whether the reservoir be full or empty.

Second.—The dam must have, at all depths, ample resistance to sliding or shearing.

In order to obtain the greatest economy, these writers designed what they called "profiles of equal resistance," that is, profiles in which the maxima pressures reached near either face remained uniform at all depths. So long as the assumed limits of pressure are moderate, say 6 to 8 tons per square foot, no objection can be raised to this method, but when we permit higher pressures—and the tendency has steadily been in this direction—some other condition must be introduced. We could

only design a profile having a uniform maximum pressure of about 14 tons per square foot at all depths, by allowing the lines of pressure in the upper part of the profile to be dangerously eccentric.

Professor Rankine has pointed out that in addition to the two conditions mentioned above, the profile of a dam ought to be designed so as to keep the lines of pressure, reservoir full or empty, practically within its center third. By complying with this requirement, we will assure at all depths a factor of safety of at least two against overturning and also assure ample strength against shearing. This condition of Professor Rankine is really the true principle upon which the design for a masonry dam should be based, the pressures near the faces being permitted to increase gradually from the top until they reach, at a depth of about 200 feet, a maximum of about 14 tons per square foot, a stress which has been safely sustained by the Almanza Dam for more than three centuries.

As no masonry dam 200 feet high has thus far been constructed, I must agree with Mr. Francis that, up to this limit of height, the crushing strength of the masonry need not be considered. But when we go beyond this and are to design a structure like the proposed Quaker Dam, then I think the limiting of the pressures, found by the usual formula, becomes the all important condition. To build a dam, 300 to 500 feet high, according to the triangular profile given in Fig. 7, Plate XX, and in Table No. 3, page 162, I think would be extremely hazardous.

It has been shown by Professor Rankine, Professor Intze and others, that so long as we have only to limit the lines of pressure to the center-third of a profile, the most economical form to withstand simply the hydrostatic pressure of the water, is given by a right-angled triangle with its vertical side towards the reservoir. As a dam must, however, have sufficient strength to resist the action of waves or shocks from floating bodies, the top of such a triangular profile must be reinforced by giving it a certain width and also a super-elevation above the highest water surface, which will depend upon local circumstances. Professor Castigliano has proposed, in the Politecnico for 1884, to convert a triangular profile, such as is given in Fig. 7 of Mr. Francis' paper, into a practical design, simply by adding the necessary top-width. While such a profile will have ample strength at all depths, it does not give the best distribution of the masonry in the wall, and, consequently, not the greatest economy.

A high dam ought to have a top-width of 15 to 20 feet. The proper manner of designing the profile is to take the given top-width as one of the data which must enter into the calculation.

If we follow this plan the theoretical triangular profile will be considerably distorted, so to speak, in the upper parts of the dam, but in the lower portions the faces will become parallel to those of the triangular form. The sections shown in Figs. 4, 5, 6 and 7, of the paper

under discussion must be regarded, therefore, as theoretical profiles, which have no great practical value.

I come now to the last topic contained in Mr. Francis' paper: The effect of the perviousness of masonry in a dam.

The experiments described are very interesting, and show certainly that the hydrostatic pressure of water can be transmitted through a certain thickness of good hydraulic mortar.

What effect will the water penetrating the wall exert upon its stability? So long as it passes through the wall and not under it, evidently none (leaving the deterioration of the masonry out of the question), for the water penetrating through any small channel will exert practically the same pressure downwards as upwards. This conclusion is verified by the experience with the Habra Dam in Algiers. When the water was first let into the reservoir the dam is said to have presented the appearance of a gigantic filter, so great was the leakage through it, and yet this fact does not seem to have affected its stability. This loss of water diminished rapidly owing to the channels of leakage becoming clogged.

After having stood for eight years, the dam was ruptured, but that was after an unusually severe storm, when the water passed about one meter high over the parapet of the wall.

Should the water penetrate between the foundation and the base of a dam, it would undoubtedly exert an upward pressure and diminish the stability of the wall.

The experience with the high masonry dams built abroad has shown, however, that with proper care and precautions, such walls, formed of ordinary rubble masonry, can be built so water-tight that only a slight dampness is noticed on the down-stream face. Taking this well-established fact in connection with the experiments of Mr. Francis, I should conclude that the water can penetrate but a short distance with its full hydrostatic pressure into the masonry, and that if it passes through a well-built dam, it loses its force, and simply goes through in a capillary manner.

It has been found that whatever leakage through the wall takes place when the water is first turned into a reservoir, diminishes rapidly, this applying, of course, also to the loss of water at the foundation.

I know of but one modern dam which has been designed on the supposition that the water pressure from the reservoir might extend under the whole base, viz.: the Gileppe Dam, built in Belgium in 1870 to 1875. This dam contains an excess of 75 per cent. of masonry over what is placed in modern French dams of the same height. I think that I am safe in stating that if the hydrostatic pressure due to the water in the reservoir could extend under the entire base of any of the recently built dams, the line of pressure would become so eccentric that the wall

would fail. In a properly constructed dam, I think, therefore, that any water leaking at the foundation is likely to have very little pressure.

If such be the case, and I think the experience with modern dams fully corroborates this view, then I should think, instead of providing drains in the foundation of a dam, as was done in the Vyrnwy Dam, or placing pipes in the masonry, as was done in the Tytam Dam, the idea being to get rid of any water that might filter into the structure, that such channels which tend to facilitate the infiltration ought to be avoided, and that efforts should be made to make the wall and the foundations as water-tight as possible.

To obtain this end, the masonry near the reservoir ought to be especially well laid, and it may also be advisable for very high dams to build a puddle bank of suitable material against the back face, for a certain height.

A. FTELEY, M. Am. Soc. C. E.—The statements expressed in Mr. Francis' paper and by Mr. Wegmann in regard to the distribution of weight on the base of high masonry dams, are founded on the assumption that the whole mass of masonry is of a homogeneous character. In fact it cannot be so. Parts of the structure must necessarily be built at different times of the year, within wide limits of temperatures; various kinds of stones must be used, which will admit of varying quantities of mortars; mortars may be made of different ingredients; differences in the sizes of the stones and in the workmanship in various parts of the structure may occur. All these and other causes will modify the local power of resistance of the masonry and the mode of transmission of strains throughout the mass.

The opinions just expressed in regard to the distribution of weight on the base of the structure are widely different. The reference made by Mr. Francis to the action of lateral support which takes place in deep natural formations, is extremely interesting, and illustrates forcibly the fact that the limits of strains recommended for walls of moderate thickness can be safely extended for great masses of masonry.

Although unable to understand why the pressures exerted on the base of high masonry dams are minima at its extremities, I find it equally difficult to admit that the maximum strains are exerted at the end of the down-stream toe. The profile of many dams shows that the masonry generally ends on the down-stream side, in a point forming an acute angle with the horizon, and too weak to support the great strains indicated by calculation; if these strains had ever obtained at that point, a vertical fracture would have occurred at a distance from the toe where the masonry has sufficient section to resist the strain.

The absence of such fracture seems to indicate the absence of excessive strains at the toe.

Between these two extreme assumptions in regard to the distribution of the weight on the base of high masonry dams, there must be a middle ground which remains imperfectly defined.

In connection with the interesting experiment described by Mr. Francis, attention may be called to the fact that the whole mass of the dam is substantially as porous as the joint which unites it to the bottom, and that the weight of the water held in the masonry must influence the results of the calculations of stability.

It may be added here that all the computations are made without reference to the adhesive power of the mortar, which adds considerably to the stability of the dam.

Mr. WEGMANN.—Mr. Fteley is undoubtedly correct in stating that the real distribution of pressure lies somewhere between the extreme views expressed by Mr. Francis and by me. As he cannot tell us, however, where that "middle ground" lies, I think we are compelled, in our present state of knowledge, to design the profiles of dams either by neglecting the pressures in the masonry entirely, as advocated by Mr. Francis, or else by calculating them in the usual way, assuming the masonry to be rigid and homogeneous. The hypothesis of rigid masonry leads undoubtedly to calculated pressures, which are in excess of those which actually exist in the wall. As regards homogeneous masonry, it is an ideal to be aimed at, although it may be never perfectly attained. I fail to see how we could obtain a satisfactory theory of dams based upon the supposition of heterogeneous masonry. Thus far all writers on the subject of masonry dams have based their formula upon the hypothesis of rigid and homogeneous masonry. The fact that the many high reservoir walls, constructed within recent years in conformity with this theory, have stood successfully, is an ample guarantee of the safety of the usual method of designing such structures.

Mr. FTELEY.—I fully agree with the last speaker as to the advisability of submitting the problem to the calculations recommended by good authorities. They establish the value of maximum strains which the structure will not probably be called upon to resist; and if the design is so proportioned as to resist those maximum strains, the assurance that the pressures will remain within safe limits is of great value.

THEODORE COOPER, M. Am. Soc. C. E.—As a listener to the preceding discussion, I agree in principle with both Mr. Francis and Mr. Wegmann.

Mr. Francis claims that the capacity of the mass is greater to resist strains at the center than at the toe. I thoroughly agree with him.

If, however, the capacity of the mass to resist compressive strain is greatest at the center and least at the toe, the point of danger is at the

toe. Mr. Wegmann is therefore correct in considering the most important point to be the toe pressure.

If the section of the dam be considered as a bracket maintained in its position by the superincumbent weight of material and resisting the overturning water pressure, we must obtain for equilibrium from the theory of equal moments certain pressures upon each element of the base which, combined with the pressure of the superincumbent weight, give the total pressure for this case and the maximum pressure on the toe.

This bracket cannot overturn without crushing the toe or shearing some portion of it off. I do not mean to say that the toe is going to take all the strain; every point is resisting, but this is the point of danger.

JAMES B. FRANCIS, Past Prest. Am. Soc. C. E.—I would like to add that we have an illustration in hydraulics. It is well known that water confined will bear a pressure only limited by the strength of the vessel containing it. Put a thousand pounds per square inch upon water in the cylinder of a hydraulic press, and there is no sensible yielding unless it bursts; it depends upon the lateral confinement. I think the same principle exists in great masses of masonry; the lateral confinement increases the supporting power; it all depends upon the strength of the lateral support.

CHARLES E. EMERY, M. Am. Soc. C. E.—In considering the strains to which the masonry at the bottom of a high dam of this kind is subject, it seems to me that we must consider the fact that the material as a whole is of a porous nature, particularly the mortar, so that the pressure of the water is communicated through the mass by a series of cells more or less connected together. If the mortar adheres at all, the tensile strength of the walls of the cells aids the superincumbent weight to resist the lifting effect of the hydraulic pressure, and the thickness of the walls of the cells reduces the area upon which such pressure acts. The latter effect must also take place to some extent when there is no adherence of the mortar, as the superincumbent weight would hold certain points of contact under sufficient compression to exclude pressure, even if those points are so small as to represent more nearly the ultimate particles, and the pressure has an opportunity to circulate between the same in small passages, like the connected cells above referred to.

Some discussion has arisen as to the resistance which would be afforded by extending the down stream face of the dam out at the bottom in the form of a toe, one gentleman hinting that rupture would take place before it offered any considerable resistance. A thin toe run out in this way would, however, deflect more easily than a thicker mass of masonry, and the moment resulting from the forces required to produce such deflection would aid in maintaining the stability of the dam. This, together with the whole

question herein discussed, in regard to the pressures upon the masonry at the front and rear of the dam, illustrates only the usual phenomena in relation to the flexure of a beam. If a post, with the upper part of the same section as that of the dam, were set rigidly in the rock, a side-pressure corresponding to that of the water, would simply bring tensile strains on the side where the pressure is applied and compressive strain on the other; and were the beam without weight, the integrated moments of strains on each side of the neutral axis would be equal and opposite. As soon as we consider the beam to have a weight equal to that of the dam, the algebraic difference of the two moments when water pressure is considered, must be the same as before. The pressure of water would simply increase the strains on one side of the neutral axis and decrease them on the other, but the arithmetical sum or algebraic difference of the integral of the moments from the neutral axis, of the increased strains on the down-stream side, and of the integral of the moments of the forces relieved on the up-stream side, would equal the moment of the overturning force produced by the pressure of water. Ordinarily the material in a dam would be so disposed that the compressive strains on the up-stream side would never be relieved entirely so as to produce tensile strains in the masonry.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—Should not the lowest part of this curve, *C V T* (Fig. 7, Plate XX), be to the right of the line *H V*, since the center of gravity of the area included by the curve should be upon the line *H V*. The center of pressure is upon that line, and it would seem that the center of gravity of any figure representing that pressure should also be upon that line.

Mr. FRANCIS.—I have not been able to find out where the center of pressure is; I have the general idea that there is a transfer of pressure from the outside towards the inside, but exactly how it would be distributed I have not been able to determine, and we have no exact data. I suppose the maximum pressure would be somewhere near that line.

Mr. BRINCKERHOFF.—Not the maximum pressure, but the center of gravity of the figure. I feel convinced myself that the center of gravity of the figure must be on the line *H V*.

Mr. FRANCIS.—My idea was that the center of pressure would be on that line, but I can give no proof of it.

Mr. BRINCKERHOFF.—No, sir; if that pressure is supported at *H*, it would be in equilibrium, and there would be no rotation; if it is supported by any number of forces whose center of gravity is not at *H*, there will not be an equilibrium.

Mr. FRANCIS.—I have based my estimate upon the excessive pressure near the outside being transferred towards the center, which is more capable of supporting it.

Mr. BRINCKERHOFF.—To assume the center of gravity of the area *C V T*

to be on the line *H V* would strongly affect the shape of the enclosing curve, and, I think, materially increase the maximum pressure.

To resist the same force of pressure applied at the same place, not only must the area of *C V T* (Fig. 7) be equal to that of *C S T*, but its center of gravity must be on the same perpendicular line *H V*. In order to fulfill this latter condition, the maximum ordinate of *C V T* must be some distance to the right of *H V*, and to fulfill the former condition, the maximum ordinate of *C V T* must be considerably greater than *H V*; in fact, the figure *C V T* will really differ much less from *C S T* than has been assumed.

Mr. FRANCIS.—The first condition of the curve is that its area must be equal to the area of the triangle. Exactly what the form of the curve would be, and where the maximum of pressure would be, I have not determined; I hope somebody else will; I have not been able to.

GEORGE S. GREENE, M. Am. Soc. C. E.—In the Yosemite Valley there is a vertical face of wall of some 3 000 feet in height. Now, how does this masonry stand all this pressure; how has it stood all that it has for ages? It is because the pressure is transmitted through the entire wall, and the base is supported against crushing by this pressure on the lower courses, resisting any lateral movement, which movement must take place if there is any crushing. And so it would be in any well-built dam. There will not be this destructive pressure upon the beds, and I don't see that we can get any pressure that will crush granite when confined laterally.

A MEMBER.—We must consider the crushing strength of the mortar; it is, of course, the weakest part of a masonry dam. It may be that a pressure of twenty tons per square foot, or perhaps one of thirty might be tried, but it would be a hazardous thing for a person to design a dam on this supposition.

Gen. GREENE.—How is it we have natural cases where a mass of masonry does stand at this height?

A MEMBER.—I should say that the crushing strength of the mortar would have to be considered.

Mr. FRANCIS.—In hydraulic presses the only limit to the weight that can be put upon the water is the strength of the vessel containing it.

Gen. GREENE.—Did Mr. Francis use thin mortar or cement? Was it filled in? Was it tamped with a ram?

Mr. FRANCIS.—It was mixed with the consistency that mortar usually is; it could not be rammed. The proportions were sand, two; cement, one. It was put in as well as I could put it in, working it with a stick.

Gen. GREENE.—I think you have had some experience as to the imperviousness of cement?

Mr. FRANCIS.—Yes; I think you can make a wall more impervious by a plastering of pure cement on the face next to the water than by any thickness of masonry as ordinarily laid.

G. Bouscaren, M. Am. Soc. C. E.—In testing small blocks of stone to destruction, if the pressure is applied gradually, failure begins almost invariably by the splitting of the blocks in a direction parallel, or nearly so, to the direction of the pressure, beginning near the faces, and extending rapidly toward the center of the block, as shown by the successive lines of division, $a\,a'$, $b\,b'$, in Fig. 1, Plate XXI. An explanation of this may be found in the fact that in all stones the ultimate resistance to tension is much less than the resistance to compression. A pressure P being applied to a stone block will create an elastic tension T at right angles to the direction of the pressure; when the limit of resistance to this tension is reached, the block will part, as illustrated, before the ultimate resistance to crushing has been actually reached.

The forces acting on all points, o, in the block (Fig. 2) are a pressure p, its reaction p', and the elastic forces t t, equal and opposite in direction, and at right angles to p; the only external forces opposed to t are the friction of faces $A\,B$ and $C\,D$ on their bearings.

Let us assume the block $A\,B\,C\,D$ to be a cube; let

a = side of cube.
$d\,w$ = element of surface of section through $m\,n$.
x = distance of $m\,n$ to face $B\,D$.
t = elastic tension per square inch.
Δ = ultimate resistance per square inch to tension.
p = pressure per square inch applied.
f = co-efficient of friction of faces $A\,B$, $C\,D$ on their bearings.

The block will part on line $m\,n$ when the sum of the elastic tensions on section $m\,n$ equals the ultimate resistance of the same section to tension plus the friction of surfaces $m\,B$ and $n\,D$.

$$\int t\,d\,w = \Delta\,a^2 + 2a\,x\,p\,f$$
$$t\,a^2 = \Delta\,a^2 + 2a\,x\,p\,f$$
$$t = \Delta + \frac{2\,x\,p\,f}{a} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

We may assume $t = K\,p$, K being a constant, hence

$$K\,p - \frac{2\,x\,p\,f}{a} = \Delta$$

$$p = \frac{a\,\Delta}{a\,K - 2\,x\,f} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

For $x = o$, $\qquad p' = \frac{\Delta}{K}$.

This is the expression of the limiting pressure (minimum) per square inch at the edges of the block.

For $x = \frac{a}{2}$ $\qquad p'' = \frac{\Delta}{K - f}$.

This is the expression of the limiting pressure (maximum) on the center of the block. For $K = f$, p'' is infinite; this means simply that for $K = f$ the block could not split in the middle, but must fail at that point by direct crushing, unless it spalls off progressively from the faces, as it generally does in fact.

From this explanation it is easy to understand why, in testing blocks to destruction, the average ultimate pressure per square inch increases with the area.

I think it is a more rational interpretation of the facts than to assume with Mr. Francis that the co-efficient of elasticity of the material increases from the faces to the center of the block.

In a wall (Fig. 3, Plate XXI) subjected to an eccentric resultant force F, having a movement around the center line $G\ G$ of its base, the maximum pressure per square inch must necessarily occur on the line of the down-stream toe B, if the co-efficient of elasticity is supposed to be constant throughout the wall.

Let e be the eccentricity of the resultant force F, and F_V its vertical component; R = stress per square inch at any point o of the base at a distance x from the center line; R' and R'' the stresses at A and B.

$a = A\ B$ = width of base in inches, the length of same Bc being equal to one.

Ω = area of base $A\ B\ C\ D$.

I = moment of inertia of $A\ B\ C\ D$.

m = moment of resultant force F around center line $G\ G$.

If the material of the wall is supposed to be perfectly elastic, with a constant co-efficient of elasticity,

$$R = \frac{m\ x}{I} + \frac{F_V}{\Omega} \qquad \ldots\ldots(3)$$

Substituting for m and I their value,

$$m = F_V\ e$$

$$I = \tfrac{1}{12}\ \Omega\ a^2$$

We have $R = \dfrac{12\ F_V\ e\ x}{\Omega\ a^2} + \dfrac{F_V}{\Omega} = \dfrac{F_V}{\Omega}\left(\dfrac{12\ e\ x}{a^2} + 1\right)$ $\qquad \ldots\ldots(4)$

For point A, $x = -\frac{1}{2}\ a$; $R' = \dfrac{F_V}{\Omega}\left(-\dfrac{6\ e}{a} + 1\right)$ $\qquad \ldots\ldots(5)$

This is the minimum value of R.

For point B, $x = \frac{1}{2}\ a$; $R'' = \dfrac{F_V}{\Omega}\left(\dfrac{6\ e}{a} + 1\right)$ $\qquad \ldots\ldots(6)$

This is the maximum value of R.

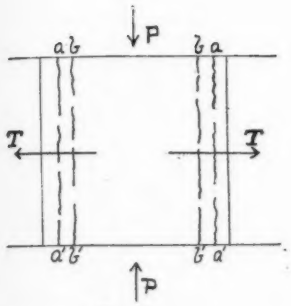


FIG. 1.

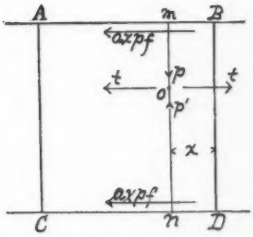


FIG. 2.

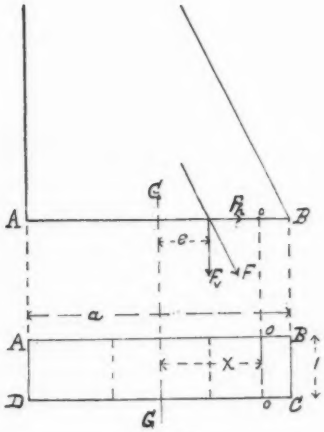


FIG. 3.

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PLATE XXI.

If we suppose the force F applied at a distance of $\frac{1}{3}$ of the width of base from the down-stream toe, $e = \frac{1}{6} a$; substituting in (5) and (6), we have

$$R' = o$$

$$R'' = 2 \frac{F_V}{\Omega}$$

The pressure per square inch is zero on the up-stream toe of the dam, and double the averages on the down-stream toe.

For $e > \frac{1}{6} a$ we would have tension at A; if the resistance to tension cannot be depended on this must be avoided, as any crack in the up-stream face of the wall, by allowing the water pressure to exert itself in an upward direction on the base, would greatly diminish the stability of the wall, hence the necessity of confining the position of F to within the middle third of the width of base.

It is difficult to understand how, under such conditions, the intensity of pressure per square inch could ever be greater in the interior of the wall than on its down-stream face, unless by failure of the material forming the face.

As long as the elasticity of the material is preserved, the maximum stress must be met with at the face, where the material being free to spall off, as has been explained to be the case for small blocks, the resistance should be assumed to be the same for both cases.

This has been the practice generally followed by designers of modern masonry dams, and it does not appear that a departure therefrom can be justified on the theory advanced by Mr. Francis.

It seems that a better way of economizing material in high dams, would be to find a mortar of greater strength than the Portland, to be used for the masonry of the face.

Joseph P. Frizell, M. Am. Soc. C. E.—The experiments of Mr. J. B. Francis, on the percolation of water through cement mortar, though the results are what might have been confidently predicted, have an important bearing on the subject of high dams of masonry. Engineers of standing have affirmed that in computing the stability of cement masonry under water, when it is bedded on firm rock, the buoyancy of the mass should not be considered, the water being excluded from the bottom by the mortar. These experiments show that such exclusion is impossible.

Another important deduction might be made. Under a dam of this description a trench of greater or less depth is sometimes excavated and filled with concrete or other masonry, supposed to be more impermeable than the body of the rock. Such a feature placed near the up-stream toe of the dam is undoubtedly an element of security. It does not, however, follow that the introduction of a second trench, further down-stream, would be an additional element of security. On the contrary,

it would be a positive, and in certain supposable conditions, a very formidable element of insecurity. Consider the case in which all the conditions are unfavorable, viz., the section of the dam substantially a right-angled triangle; the bed rock seamy; one trench along the up-stream toe, the other along the down-stream toe. Add the supposition that the up-stream barrier is defective, the down-stream one tight, the whole bottom of the dam would be exposed to an upward pressure nearly equal to that due the head, and in the aggregate approaching the total weight of the dam. This would be a combination of circumstances not likely to occur in practice. Nevertheless, as suggested by Mr. Francis, no dam of this description, as ordinarily constructed, can be wholly free from action of this kind. The practical disposition suggested by these considerations, is to limit the strictly water-tight construction, both above and below the surface of the rock, to a few feet in thickness of the up-stream face, and make arrangements for free egress to all water that passes this barrier.

Such an arrangement would have advantages other than statical, which would become apparent in the course of time. The down-stream face of a high reservoir dam is fully exposed to the action of frost, which, as every one knows, acts much more rapidly in the disintegration of rock, when the latter is saturated with water, than when it is dry.

E. Sherman Gould, M. Am. Soc. C. E.—A very able and important paper. The observations upon the crushing of stone masonry are excellent. There can be no doubt—indeed the proof rests upon direct experiments—that resistance to crushing increases with the size of the block crushed.* It may be remarked in passing that this increase of resistance does not seem to hold good for tensile strains, for Mr. Clarke, in his experiments on the Boston Main Drainage Works, found no appreciable difference in the elementary resistance of cement briquettes of different breaking sections. Since the mortar is generally the weak part of masonry structures, it would appear that this should be tested, instead of the stone. Also, that the specimen cubes should be kept under sustained pressure for several consecutive days. No doubt the material would show very much lower results, both in tension and compression, under this treatment.

As Mr. Francis well points out, any force impeding lateral expansion retards rupture from pressure. A familiar application of this principle is the ringing of pile heads to prevent splitting under the blows of the driver.

Several practical consequences result from these facts: It is evident, as Dupuit points out, that a large stone supporting the foot of an iron column of much smaller section, will sustain a much greater pressure per square inch than a smaller stone of equal section with the foot of

*This was written before seeing the Watertown experiments, which I do not touch upon here, as Mr. Francis refers to them at the close of the discussion.

the column and entirely covered by it. It is also, I think, reasonable to suppose that a very deep foundation wall, particularly if built with vertical faces, and closely imprisoned by compact earth, is less liable to crush under pressure than a similar wall standing in relief upon the surface of the ground. Also, the exposed bed-arrises of ashler face work will be more likely to check if dressed to a sharp right angle than if worked to a pitched draft with a rock-face projecting immediately beyond it, so as to form an obtuse angle, and thus buttress the arrise, as it were, by an enveloping mass.

The following remarks upon this subject, translated from the treatise on stone bridges, forming the 10th fascicule of Mons. Debauve's Manuel de l'Ingénieur, will be heard with interest:

"Crushing is due, as Dupuit remarks, to the lateral expansion of the stone, the compressed parts endeavoring to escape at the sides, where they find no impediment to so doing. When the pressure becomes so great as to destroy the cohesion of the fibres, they burst asunder, and crushing ensues.

"Any cause which opposes this lateral expansion will have for effect the retardation of the crushing of the material, by annulling, more or less, the tendency to spread. Mons. Pelletreau, in a recent paper, corroborates the above statement, by demonstrating the following proposition: Other things being equal, a prism of stone resists crushing in proportion to the degree of friction existing between its base and the surface on which it stands."

The case of the sustaining power of the top and sides of the excavation for the Chicago tunnel, could no doubt be fortified by many similar instances. It appears to me that in the case of the larger tunnel, we have an opening 9 feet wide, spanned by an arched girder of stiff clay with horizontal extrados, 28 feet thick at the centre, fixed at both ends, and sustaining a nearly uniformly distributed load of about 6 000 pounds per running foot, including its own weight.

The formulas given by Mr. Francis for the proper length of base of a theoretical triangular wall, are very good and useful, particularly No. 12, which gives this dimension under the most unfavorable conditions of pressure.

In speaking of the intensity of crushing strains, Mr. Francis refers to the well-known formulas for calculating the pressures upon the two toes of a reservoir, when the resultant does not cut the base in the middle. These formulas are given, with considerable development of the principles upon which they rest, by Mons. Debauve in the work named above, and are also demonstrated, he states, by other methods by Mons. Bresse in his *Cours de Mécanique Appliquée*. They are founded upon certain assumptions of homogeneity and rigidity, which we know cannot be fully justified by facts, but without which the problem could not be dealt with mathematically. Indeed it may be asked whether we

do not concede all that these assumptions demand when we admit—as I believe we always do—that the elementary, or unit strain is uniform throughout the base of a wall, symmetrical about a vertical axis.

Practically, I think we shall find that profiles of equal resistance, determined by these formulas, commend themselves to the eye as possessing strikingly appropriate proportions; and, when joined to a moderate limiting elementary pressure, have given rise to structures which have proved themselves eminently fitted for the purposes for which they were built.

The views of Mons. Debauve himself upon this subject are expressed as follows, in a letter which I received from him a few years ago: "These formulas should be employed with great discretion, since they assume that the masonry is homogeneous, and take no account of its resistance to traction. Their use should therefore be restricted to cases where the resultant of pressures does not move greatly from the middle of the base, and we should always endeavor to bring this resultant into the neighborhood of the middle of the sections, as is done in the profile of large masonry dams."

If I understand Mr. Francis aright, he advances the proposition that if a dam, of no matter how great a height, be properly proportioned to resist overturning, it cannot fail either by sliding or crushing. The French authors, on the other hand, as far as I am aware, test for sliding, but take no account of overturning, thus tacitly asserting that if the wall be safe as against crushing, it cannot overturn. Perhaps we have here only two different ways of saying the same thing, in which case Mr. Francis' way of putting it is certainly the simpler.

In connection with the question of upward pressure upon the base caused by water running through fissures in the rock, Mr. Francis suggests that drains might be constructed in the base of the dam to carry this water away. Mr. Francis intimates, however, that they might be objected to, upon certain grounds. I think they would be highly objectionable features in a structure, the very nature of which should be unbroken compactness. If sufficiently large and numerous to be of service in the way intended—and this would have to be guessed at in their construction—they could not fail, if the feed were considerable, to occasion a very large loss of water, as they would constitute just so many important leaks in the dam. Moreover, I do not see that such drains would prevent the communication of upward thrust (although they might diminish it by changing hydrostatic to hydraulic pressure), unless all the water liable to reach the base in this manner were caught in drain-pipes sufficiently strong of themselves to resist bursting pressure, which would be manifestly impracticable.

As regards the experiments upon percolation and pressure transmitted through mortar I cannot say that I consider them satisfactory. I do not think the apparatus used was one fitted to give trustworthy results, even if

the leakage mentioned did not exist. I do not think that the removing the varnish from the inside of the reducer, and inserting the central pipe, were sufficient guarantees that the water did not pass around, instead of through, the mortar, at least to a considerable extent.

I think it may be doubted if the concluding paragraph of Mr. Francis' paper, regarding the loss of weight of foundations carried below the permanent level of water, is to be taken as an unquestioned fact. It would of course be safest, in such cases, to make calculations on both assumptions, and take the result which indicated the heaviest work.

W. W. Maclay, M. Am. Soc. C. E.—Some experiments recently made, as a member of the Committee on Compressive Strength of Cements, may be of interest in relation to that part of the paper of Mr. Francis, giving his experiment and deductions on the transmission of pressure through hardened cement mortar.

A brick of Portland cement mortar made by weight, 1 part Portland cement, 2 parts standard sand (crushed quartz passed between sieves of 400 and 900 meshes per square inch), one year and two weeks old, was placed in Fairbank's machine, at No. 88 Thomas street, of this city, used for forcing water through building brick.

Under a pressure of 75 pounds per square inch, kept up for six hours, the brick was practically water-tight except that, towards the end of the six hours, the packing leaked a little, and when this was tightened the leak stopped.

The portion of the bottom surface of the brick not covered by the packing, and exposed to the water pressure, was a rectangle $1\frac{1}{4} \times 5\frac{1}{4}$ inches.

A brick of Portland cement mortar, 1 part cement, 1 part sand, one year old, fifteen minutes under pressure of 75 pounds per square inch, showed no signs of leaking.

Bricks of Portland cement mortar, 1 part cement, and 3 and 4 parts sand, one year old, both leaked freely under hardly any pressure, the gauge indicating less than 1 pound per square inch.

F. Collingwood, M. Am. Soc. C. E.—Some of the experiments made at the Watertown Arsenal, bear out the statement made by Mr. Francis in the paper just read, viz., that the central portion of a block is what gives the greater part of the strength to resist compression. Taking the National Portland cubes, mixed 1 to 3, and tested at forty-six months' age, we find at 200 pounds pressure per square inch, 8-inch cubes, compressing an average on 12 inches, length of 0."0048, 12" cubes ."0016; 16" cubes ."0011; at 700 pounds, ."0102, ."0052, and ."0040, etc.; the strengths at rupture being 2 478, 2 434 and 2 520 pounds, respectively. When mixed, 1 cement, 3 sand, 6 broken stone, at 200 pounds, the com-

pressions were for same sizes, respectively, .″0054, .″0028 and .″0015; at 700 pounds they were, .″0100, .″0072, .″0046, and the strengths at rupture, 3 027, 2 690 and 2 979 pounds.

The cubes of neat Portland give the same results as to strengths; twenty-two months cubes, 8, 9, 10, 11, 12 inches, give averages at rupture of respectively 4 781, 4 745, 4 761, 5 381 and 5 439 pounds per square inch, but the compressions do not run so regularly.

Brick piers laid in Portland cement mortar, mixed 1 to 2, do not follow the law. An 8-inch pier, 16 inches high, fifteen months old, ruptured at 3 776 pounds, and one twenty-three months old, 80 inches high, 2 249 pounds per inch square.

12-inch piers, twenty-four months old, 120 inches high, broke at 2 000 to 2 250 pounds; a 16-inch pier, twenty-four months old and 121 inches high, at 1 887 pounds. The compressions were approximately uniform for all, and as shown, the small piers were as strong as the large.

The experiments on these piers show that the compression does vary quite uniformly with the pressure, nearly up to the point of rupture. and there will be undoubtedly, as stated, a tendency for the pressure in a dam to approach uniformity in various parts of a given section. Before the pressure at the toe could reach double that at the up-stream face, the compression would cause a yielding, and the interior would have a greater portion of the pressure thrown upon it.

The experiments on transmission of pressures through the mass seem to be conclusive, and certainly point to the necessity of taking account of the lifting force of the water. Of course the amount of water penetrating into the masonry, and held there, at any one time, adds its weight to that of the masonry, and is, in so far, an element of safety; but this is small. Any means by which the head can be destroyed would remove the pressure; and this leads to interior drains, such as have been provided in the Vyrnwy Dam. But without careful work in construction, such drains might prove of themselves an element of danger.

While the portion of a dam below the level of ground-water may be treated as a foundation merely, it is undoubtedly a part of the dam, and it would seem should be so considered, although requiring a modification in whatever formula is employed, if all circumstances be taken account of.

The level of ground-water below the dam is likely to be a variable quantity, and in time to come might be decidedly changed.

The conclusion that might at first glance be drawn from this experiment would be that in every case the full pressure due to a column of water having a base nearly equal to that of the dam and a height equaling that in the dam, would be acting to overthrow the dam, in addition to the horizontal pressure of the water against it. This cannot be true

unless the lower face of the dam can be made absolutely impermeable to water. Under actual conditions (supposing a concrete dam) the water will penetrate horizontally throughout the mass, and the pressures at a vertical plane at any given distance from the inner face of the dam will all bear a constant ratio to those at the inner face; that is to say, a "free surface," or zero at the top, increasing to a maximum at the bottom, but with no unbalanced upward reaction, unless the strata underneath the dam be more permeable than the dam itself. The pressure at any point will be, instead of that due to the height (h), that due to $\frac{h}{r}$, and will diminish as the distance of the vertical plane from the inner face increases. It becomes zero at the plane where the water leaves the mass, in this case probably by evaporation.

Now, in just so far as the dam is less permeable at some points than others, or in so far as there are direct channels underneath, or a more permeable material underneath, will there be an unbalanced upward pressure, which must be considered.

We have, therefore, the two limits: either the full upward pressure acting, which would certainly be an extreme case, on the one side, or no unbalanced pressure, which is probably as extreme on the other.

E. A. FUERTES, M. Am. Soc. C. E.—I can discuss, in a very brief manner only, the paper of Mr. Francis on the form of "High Dams." As is to be expected from this distinguished engineer, his paper has an important and useful practical bearing. His views on this difficult subject are entirely correct in most respects, and probably in all respects, if the ambiguity resulting from some of his conceptions of the elastic limit of stone could be explained by defining the sense in which he deals with this property of matter. In order to avoid falling into error on account of an insufficiently precise definition of the "modulus of elasticity," as applied by Mr. Francis in this case, I have consulted with my colleague, Professor Church, whose familiarity with mechanical laws makes him one of the highest authorities on this question. Mr. Francis is correct, according to Professor Church, if he defines "modulus of elasticity" as "the quotient obtained by dividing the stress per unit area at any instant, by the contraction per unit length which the stress has produced, irrespectively of whether the elastic limit has been reached or not."

Under the above definition, and granting that the compressing plates do not change their form, and remain parallel as they compress the specimen, Mr. Francis' deductions are entirely accurate; and the "modulus of elasticity" of the stone increases from the edges towards the center. Therefore, in order to produce uniform contraction at all points, the central pressure must be greater at the center than at the edges; and, consequently, the average pressure per unit surface ob-

tained by testing specimens of various areas will be greater as the area subjected to compression increases. Mr. Francis is also correct in ascribing the result to the support which each elementary supporting column derives from its neighbor; which support must become a maximum at the center of the specimen compressed. Without wishing to be hypercritical, I will say that the transverse section of dams employed by Mr. Francis, as representing the form most economical of masonry, is not strictly correct, though it is admirably suited for the points he wishes to make, when proper deviations from this form are made to suit the cases of variable pressures and changes due to time, accumulation of material heavier than water on the up-stream face, and the widely different conditions of the foundations of dams.

It is true that in reference to all the mechanical conditions of stability, similar relations exist at each horizontal stratum of any dam; but in the triangular dam the area of these strata are proportional, arithmetically, to their distance from the top of the dam, while the resultant pressure diminishes as the square of this dimension; and, therefore, the triangular section gives an excessive cross-section against crushing in the upper beds of the dam.

Mr. Wegmann's recent book on the Quaker Bridge Dam (which is as yet the best thing written on this subject in any language) develops an exact method for determining the profile of dams on the accepted trapezoidal formula, and in a manner entirely simple and practicable.

The following criticism is made by Professor Church: "The conception of a factor of safety, as inferred from the paragraph on page 160, seems very unusual since it ignores the existence of an elastic limit in dealing with materials under stress; also, further on, it is stated that by giving the triangular wall a base of such length as to cause the resultant pressure on the base to pass through the down-stream edge of the middle third, a factor of safety of 'two' is secured; because the moment of the weight of the wall about the down-stream toe is double the value it would need to have if the resultant pressure passed through that toe. Now, in the latter case the wall would be on the point of overturning, and a (theoretically) infinite intensity of pressure would be produced at the toe, instead of a pressure merely double the maximum intensity occasioned in the first case; that is, the use here made of the term 'factor of safety' seems peculiar and misleading, since we generally consider the factor of safety to be the ratio of the breaking stress to the highest permissible actual stress. With the usual definition of the factor of safety, *eq.* (13) and *eq.* (14) of page 161, are evidently not in accord."

Much discussion has been evoked on the subject of founding bridge piers, as affected by the buoyant effort of the immersed masonry; and Mr. Francis' ingenious apparatus does justify the expediency of fully taking into account the complete permeability of any dam or pier as a

useful precaution to offset the undiscoverable defects of building, when we are confronted with the impossibility of even guessing at the extent of such buoyant effort. But it cannot be admitted as a certainty that "an upward pressure may be transmitted through the mortar to the entire base" of the dam, for such a pressure could be transmitted only when the entire base is lifted from its support; and in this case no pressure could be exerted on the bed of the foundation. So long as the specific gravity of the material of the dam exceeds that of the water under the ordinary conditions of dams, direct contact and pressure must exist at many points of the base, however irregularly distributed or restricted they may be to small and unconnected areas. This fact points to the necessity of providing for increased surfaces, to diminish the probability of overloading the points of contact, thus increasing the probability of making them more numerous.

Experiments are now under way at the cement testing laboratory of Cornell University to ascertain the change in the law of pressures due to change in the linear dimension of the specimens subjected to compression; and with other valuable results already obtained in reference to this important subject, the entire investigation will be submitted to the Society.

Regretting that pressing engagements prevent me from extending this discussion to many other interesting points in Mr. Francis' valuable contribution, I will close these hasty notes by referring those who may wish to follow the argument of Mr. Francis on the subject of the formation of a plate-band in the material above the Chicago Tunnel, to Rankine's "Civil Engineer," pages 212 and 435. This subject is in a very unsatisfactory condition, and the novel aspect (to me) in which Mr. Francis presents his views, may open a new avenue for attacking this obdurate problem. I can see that in very solid rock a certain thickness above the top of a tunnel may act like a built-in beam resting upon the span made by the excavation; but in clay (more or less plastic) a more or less completely closed arch must be formed.

JOHN W. HILL, M. Am. Soc. C. E.—In low dams, if the profile and weight of the masonry is sufficient to prevent sliding on the base, and overturning, there will be no danger from excessive pressure at the outer toe. In high dams, however, the conditions of stability and resistance to overturning may be had with a given profile and weight of masonry, but with dangerous pressures on the outer and inner toes; hence, the advantage of the Debauve method, quoted by Mr. Francis, which makes the test of strength, the pressures developed at the toes of the structure, in calculating the stability of high dams. If the maximum pressure at the base, or at any joint, *i. e.*, the pressure at the outer toe with the reservoir filled, or at the inner toe with the reservoir empty—falls within a limit (to be fixed), for the materials used in construction of the work, all other conditions for safety are necessarily fulfilled.

It has long seemed to the writer that the problems associated with masonry dams, not subject to a flow over the crest, were less complex than with dams subject as the Vyrnwy Dam for the Liverpool Water-works to periodical overflow; or as low dams placed in the channel of a river, subject to complete submersion. In the former the strains are divided into; (1) the pressure on the inner toe, with the basin or reservoir empty, and, (2) the pressure on the outer toe, with reservoir filled for high dams; and (1) the resistance to sliding on the base, and (2) resistance to overturning with water at the crest for low dams; while with dams constructed for an overflow over the crest, or submerged at flood, we have these strains to consider in addition to the variable pressure upon the face of the masonry, developed by the approaching flood, and the overflow of variable depth on the crest.

During the past season the writer had occasion to make plans for a low dam, submerged at flood, to be constructed in the channel of the Blanchard River, for the Findlay, Ohio, Water-works. The purpose of the dam was to create a storage of water in the channel of the stream above and in a reservoir adjacent to the river, about 1 000 feet up-stream from the dam. The reservoir is an artificial work, constructed in a natural depression, and the channel of the river immediately above the dam will be widened, by excavation of the soil of the banks to the natural rock (which constitutes the bed of the stream), for a distance of 1 500 feet above the dam. The slope of the bed of the river is 3.159 feet per mile, and the influence of a dam 8 feet high is felt for a distance of 2.53 miles up-stream. The storage provided in the artificial basin and the channel of the river is nearly 90 000 000 gallons.

In designing the dam, that is, in determining the heighth and length of overfall, two general objects were kept in view; (1) to produce the largest storage of water consistent with the safety of property above, and (2) to provide for the discharge of the flood volume, with the least increase of surface elevation of the water over the dam. The natural channel of the river, and the topography and value of the property at and near the dam site, and the cost of the structure, of course, were always modifying factors.

Calculations based upon the best obtainable rainfall data, the area of the water-shed of the stream above the point where the dam is located, and the topographical and geological character of the drainage ground, placed the ordinary flood discharge at 9 180 cubic feet per second. The overfall was accordingly made 200 feet long, and to the coping of the wing-walls, or roots of the dam, the depth was made 6 feet. That is, the clear water-way over the crest and between the wing-walls is 200 x 6 feet. The coping stones on the crest of the dam are bush-hammered on the upper beds, and inclined—with the lower edge up-stream—one foot in eight.

The clear water way of 1 200 square feet, within the wings of the

structure, according to Mr. J. B. Francis, has a flood capacity of

$$Q = 3.012\ L\ h^{1.53}$$

where L = length of overfall,
h = head over crest of dam; and
Q = discharge in cubic feet per second;
hence $Q = 3.012 \times 200 \times 6^{1.53} = 9\,342$ cubic feet; or slightly more than the assumed flood.

The waste way over the crest of the dam is not proportioned to an extraordinary flood, for from high water marks found along the river, and information obtained from farmers living for many years upon land adjacent to the stream, the writer assumed the extraordinary flood, which occurred February, 1884, as roughly 20 000 cubic feet per second, or more than twice the ordinary flood discharge.

The highest water mark (near the site of the dam), above the lowest elevation of bed rock in the channel is 17.96 feet; the cross-section of the natural channel at an elevation of 8 feet (crest of dam) above same datum was 946.48 square feet; the surface width of stream, at extraordinary flood tide, was taken as 500 feet, and the increased elevation of high water, with the obstruction created in the channel by the dam, was taken as 1.89 feet. The cross-section of the improved channel of the stream to the crest of the dam is 1 300 square feet, and the section of clear water way above the coping stones of the wing walls was taken as 1 400 square feet.

From these data the unbalanced pressures on the face of the dam—per foot of length—were calculated as follows:

Water at crest	2 000 pounds.
Water 6 feet over crest	4 582.2 "
Water at extraordinary flood, 11.85 feet over crest	3 990.0 "

The resistance to sliding, on the base, was calculated as follows:

$$R = \frac{c\ w\ f}{p}$$

where c = volume of masonry per lineal foot of dam, in cubic feet = 78.25;
w = weight in pounds of masonry, per cubic feet = 144;
f = co-efficient of friction, taken as 0.60;
p = unbalanced pressure, in pounds, on the face of dam = 4 582.2; and
R = resistance to sliding, in terms of unbalanced pressure (pressure taken as unity), factor of safety.

Inserting values: $R = \frac{78.25 \times 144 \times 0.6}{4582.2.} = 1.475.$

The bed of the river is solid rock (limestone) of the Niagara series, nearly level from end to end of the dam, and is cut, as shown by the accompanying sketch, Plate XXII, to receive the footing courses, and the resistance of the structure to sliding, instead of being 1.475, based on smooth continuous surfaces, is really equal to the cohesion of the rock in the bed of the stream and the dam.

The resistance to overturning, in terms of the unbalanced pressure on the face, was calculated as follows:

$$L = \frac{c \ w \ x}{p \ y}$$

where x = horizontal distance, in feet, from center of gravity of section of dam to outer toe = 7.2;

y = vertical elevation, in feet, of point of application of pressure to face of dam = 5.03;

L = resistance to overturning.

Inserting values:

$$L = \frac{78.25 \times 144 \times 7.2}{4582.2 \times 5.03} = 3.52.$$

Testing by the Debauve method:

$$p' = \frac{2\,(2l - 3u)\,P}{l^2}$$

where l = length of joint, or width of base = 12 feet;

u = distance from outer toe to point of application of pressure, p = 5.17 feet;

P = resultant of vertical and horizontal forces = 11 268 pounds; and,

p' = pressure in pounds per square foot at outer toe.

Inserting values:

$p' = 2\ [2 \times 12 - (3 \times 5.17)]\ 11\,268 \div 12^2 = 1\,328.685$, or 9.227 pounds per square inch.

There is one element in the construction of high masonry dams, and perhaps other structures, which Mr. Francis does not mention, viz., the importance of a solid, unyielding foundation. Even if the profile be correct, and the pressures at the toes of the base be within the limits given by the Debauve method (92.5 to 114 pounds per square inch), settlement due to the vertical component of the forces developed at the toes may ruin a structure of this character.

The lower silurian formations, and often other series of stone in parts of the country, are in layers or ledges separated by seams or strata of clay, which, when moist, will yield and flow almost like a lubricant; and before placing an important work like a high dam or retaining wall, upon such a foundation, careful borings and drillings should be made in the subterranean formations. Should the strata of the foundation be separated by seams of clay, the probability of its becoming softened and

forced out under pressure is necessarily increased, when the structures over it are a high masonry dam and a deep reservoir of water.

A settlement which would have no dangerous effect on the foundation of a building, might threaten the integrity and usefulness or even the safety of a high dam.

The main retaining wall of the Eden Reservoir of the Cincinnati Water-works failed from this cause. This wall has a height at the center of the ravine across which it is placed of 118 feet and a width of base of 48.5 feet, and is cracked on the face in several places, the cracks having a generally vertical direction, and sensibly widening from the top downwards. This wall is built of a compact limestone, cost nearly $800 000, and was built in the usual "happy-go-lucky" manner of city works.

Taking the highest limit assigned by M. Debauve, or rather the pressure developed at the outer toe (with reservoir filled), for the Ban (France) dam, the load on the foundation becomes $113 \times 144 \div 2\,000 = 8.14$ tons per square foot, while M. Krantz, chief-engineer of bridges and highways, France,* puts the limiting pressure or load per square foot at the toes of high dams at 6.12 tons, and after recounting several disasters with high masonry and other dams, says: "In view of such chances, the engineer must not be too bold, nor take upon himself before the public a responsibility which is powerless to repair such disasters. However little it may lean toward rashness, boldness in such cases may become almost a crime. * * * * In my opinion it is better not to build reservoir walls if we have not the necessary resources to build solidly, than to build carelessly at the risk of frightful catastrophes. On all these accounts, I think that in calculating strength the pressure of 6 kilogrammes per square centimeter (85 pounds per square inch), should never be overstepped."

Mr. George F. Deacon, borough engineer of Liverpool, however, upon investigating the plans for the Vyrnwy dam, found the pressure at the inner toe, with reservoir empty and a wind pressure on the back, of 40 pounds per square foot, to be 8.7 tons; and with reservoir full, and a wind pressure of 60 pounds per square foot on the exposed face, a pressure or load of 6.36 tons on the outer toe.

The foundation of this structure is "on rock belonging to the Caradoc group of the lower silurian system." Mr. Deacon, in his report to the corporation, says the dam "has an exceptionally high factor of stability."

The facility with which water under moderate pressures will percolate through brick, sandstone, unglazed stoneware and porcelain, is taken advantage of in the construction of our best known domestic filters. The "Pasteur" filter is constructed with tubes about 1-inch in-

* "Study on Reservoir Walls," by J. B. Krantz, Ingénieur des Ponts et Chaussées, Paris. Translated by Captain F. A. Mahan, U. S. A., 1883.

Masonry Dam
Findlay O. Water Works

Stability by Depaure Method

$$p = 2(2l - 3w)\frac{P}{l^2}$$

$$p = 2\left[2\times 12 - (3\times 5.17)\right]\frac{11268}{12^2} = 1328.685$$

and $\frac{1328.685}{144} = 9.227$ pds. per sq. in.

Limiting pres. 92.5 per sq. in.

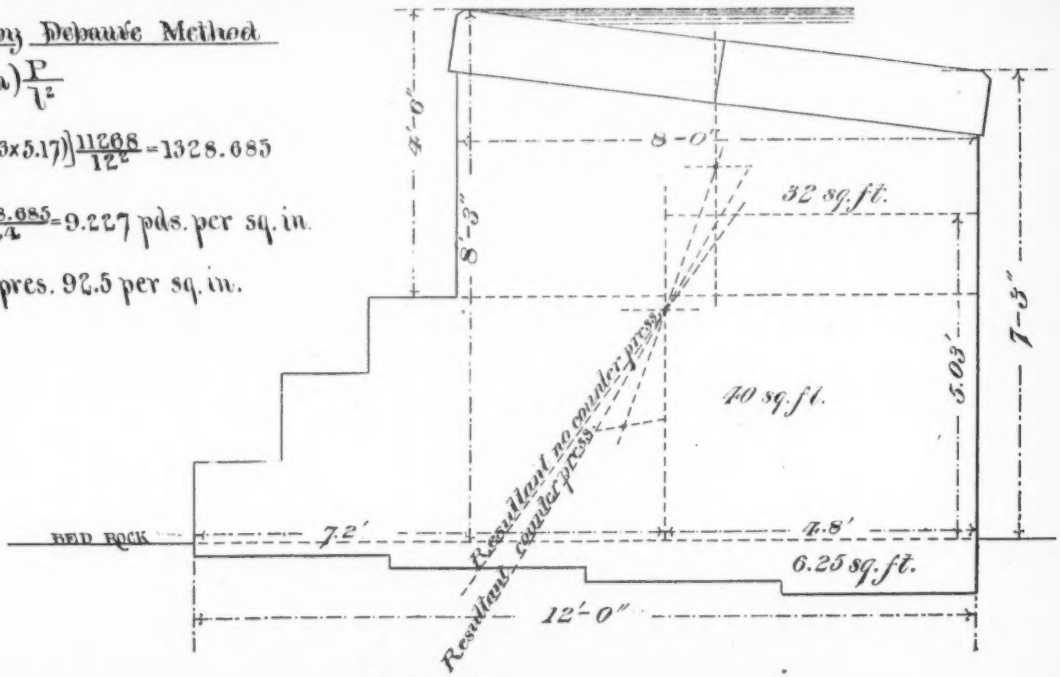

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PLATE XXII.

ternal diameter and $\frac{3}{32}$ inch thick, of unglazed French china clay, and under a pressure of 25 pounds per square inch, seven such tubes, about 9 inches long, will filter all the water required for drinking and cooking purposes by a family of eight or ten persons.

The writer has known of instances of earthen reservoirs, the inner slopes of which were paved with two courses of hard burned brick, set in cement mortar, leaking badly, even after the application of successive courses of strong cement grout over the face of the brick work. An engineer, well known in the water-works line, after constructing a small reservoir, in which the puddle was laid on a slope of one and one-half to one, and carelessly placed and tamped, attempted to make the basin water-tight by a layer of cement over the brick paving, with no success. The paving in this instance was in two courses, the lower of bricks, set flat on a bed of sand, and the upper of bricks on edge, set in neat Portland cement.

In Mr. Francis' experiment upon the perviousness of cement mortar under pressure, it is to be regretted that his tests were limited to one thickness and one pressure; tests made on different thicknesses of mortar, or cement slabs, and at different guaged pressures, would have furnished the data for an expression showing the relations of pressure, thickness, and rate of percolation. Cisterns built of brick in cement mortar, are never proof against infiltration of water from the surrounding soil, and thin foundation walls of buildings, if the soil be damp, are frequently covered with a sweat upon the inside. Thick walls, of well built masonry, when the thickness is 5 feet or more, are often dry upon one side while in contact with water upon the other; which suggests that there is a minimum thickness for certain materials, through which water will not pass by percolation, even under considerable pressure. This thickness, of course, must increase in some ratio (unknown to the writer) with the pressure.

Mr. FTELEY.—In connection with the proposed construction of the Quaker Bridge Dam for the additional water supply of New York—a structure of unusual magnitude—opposite opinions have been expressed by engineers in regard to the advisability of adopting a straight or a curved line for the plan of high masonry dams. Any remarks on this subject would, without doubt, be received with a great deal of interest.

Mr. FRANCIS.—It is a matter I have given some thought to, but have made no calculations upon. We can test it, however, by the consideration of extreme cases.

First.—If the dam is built in the form of an arch, and depends for its stability entirely upon the transfer of the pressure against it to abutments, similar to an ordinary bridge; when the pressure is brought against it there will be a yielding corresponding to the settling of an

arch when the centers are struck. If the dam is long, say a thousand feet, the amount it will yield will obviously be considerable; perhaps, in the middle in a high dam, a foot or more. Being built on the rock, which cannot yield laterally, this would cause a sliding on the base or fractures in the masonry, most probably the latter, which would be a fatal objection.

Second.—If the dam has a section sufficient to resist the pressure of the water at every point without yielding, independently of lateral support, if built in the form of an arch it can get no additional strength from it, as the support due to the arch cannot come into play until there is some yielding to the pressure.

According to my view, the two forms of resistance are not homogeneous and cannot be made to act harmoniously together.

N. M. EDWARDS, M. Am. Soc. C. E.—I should think that in large masses of masonry in high dams, the question of cracks would be important in determining that the dams be statically strong.

In fissures, either great or small, hydrostatic pressure would have its power in proportion to their extent, and tend to disintegrate and destroy the structure.

Even if it were possible to build it in a solid mass, a crack or cracks from a fault in the earth's crust, might enter in, and thereby the masonry be resolved into a broken mass.

Mr. FRANCIS.—The experiments on the increased resistance per square inch to crushing depending on increase of surface, which I have quoted from General Gillmore's experiments made in 1875, were limited to hand specimens by the limited capacity of the hydraulic press with which they were made, which did not exceed fifty tons. The experiments have since been extended with the testing machine at the Watertown Arsenal, near Boston, which has a capacity of four hundred tons. The results of the experiments made with this machine on cubes up to twelve inches on the side and other forms, and of various materials, have been published during the current year.* They do not bear out the results of the great increase of resistance per square inch to compression deduced from the previous experiments on the smaller cubes, the accuracy of which is, however, not questioned. The causes of the differences are explained by General Gillmore in his summary, as follows:

"The law of increase of compressive strength per square inch of bed-surface, with increasing size of cubes, which was based upon experiments made some ten years ago with various but limited sizes of Berea sandstone, was not confirmed when larger cubes of Haverstraw

* Notes on the Compressive Resistance of Freestone, Brick Piers, Hydraulic Cements, Mortar and Concretes. By Q. A. Gillmore. New York: John Wiley & Sons, 1888.

sandstone, cement, mortars and concretes were tested. That some such law exists for cubes within certain limits, cannot be doubted, not only in view of the Staten Island experiments, but of experiments made by foreign investigators, referred to in this report. The failure of the law with larger cubes seems to be due to the lack of the homogeneity throughout the mass of such pieces; this is indicated by the strain diagrams. It is only possible to discover defects in a large piece by dividing it into smaller pieces; and when the most perfect of these fragments are selected to prepare small test-samples, approximately true unity in regard to homogeneity of structure may be obtained. It is thought that large cubes are not such units, or true monoliths; that they represent a species of conglomerate of smaller irregular pieces, bound together by a cementing substance of varying strength, and perhaps partially separated by minute cracks and cavities. With cements, mortars and concretes, the relative amount of work expended in consolidating the material in the moulds cannot well be evenly distributed or proportioned for all sizes of cubes; the amount of set developed in small and large cubes of the same age is undoubtedly different. This is probably the reason why in all of the cements, mortars and concretes the smallest sizes of each series of cubes proved the strongest per square inch of surface pressed."

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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THE CONSTRUCTION OF THE SWEETWATER DAM.

By JAMES D. SCHUYLER, M. Am. Soc. C. E.

READ OCTOBER 17TH, 1888.

WITH DISCUSSION.

The question of an adequate water supply for irrigation, as well as for the domestic use of cities and towns, is one which, in San Diego County, California, necessarily involves storage reservoirs. The streams of the county are of an intermittent character. The mountain ranges in which they head and from which they flow to the coast do not generally exceed 6 000 to 6 500 feet in elevation—an altitude too low in the latitude of San Diego to maintain perpetual snow upon their summits, or even to retain such proportion of the winter precipitation as comes in the form of snow (not usually more than ten per cent.) for more than a few days or weeks. As a result the streams are torrential in winter and carry large volumes of water, but in summer and fall, when most needed for irrigation, are almost dry for twenty or thirty miles of their lower course, with the exception of certain seasons of such unusual

rainfall that no irrigation is required—seasons that come at rare intervals. Ordinarily the streams in summer reverse the usual order of nature, and are largest at the small end, and to get a water supply the engineer must either go far back into the mountains and gather together a number of small living streams and springs, and pipe them out long distances, or construct dam and storage reservoirs to retain the winter floods. Fortunately nature has compensated for the existing conditions by providing numerous favorable sites for such constructions, and every stream of importance in the county has available sites for storage dams of large capacity. A number of water companies are engaged in preparing for extensive works of this character, which, when completed, will provide irrigation facilities for several hundred thousand acres of land otherwise unproductive.

This era of development was inaugurated but recently, and the first completed work of the character is the Sweetwater Dam and Reservoir and the extensive pipe system reaching out from it.

The circumstances which led to the building of the dam were that the San Diego Land and Town Company (a first cousin of the Atchison, Topeka and Santa Fé Railway), owned a large body of fertile and desirable mesa and valley lands bordering on San Diego Bay, adjacent to San Diego on the south, which were unsalable without water to irrigate them. These lands constitute the greater part of the "Rancho de la Nacion," including the town site of National City, which also languished with thirst. The Sweetwater River passes nearly through the center of the lands, and is of the nature described—intermittent in flow, at least for many miles above its mouth.

The first storm or two of the rainy season is absorbed by the thirsty earth, and the stream generally does not begin flowing into the bay until late in December, or in January. After each heavy storm thereafter its volume will reach 500 to 1 000 cubic feet per second, for a few days, and within a fortnight recede to 10 cubic feet per second.

The last severe storms of the rainy season are usually in March, and the flow of the stream will generally dwindle to one or two cubic feet per second by June 1st, which amount may be maintained through the remainder of the year, but not always. The large supply running to waste each year, followed by months of scarcity, naturally suggested storage, and the first cañon above the mouth of the stream was selected as the place to accomplish the object. This narrow gorge, 7 miles east

Plate XXIII.—Sweetwater Dam.

of the bay, is a deep and narrow cut, half a mile in length, through a dyke of trap rock or trachyte that intersected the valley of the Sweetwater, leaving above it a broad level valley some 3 miles long, ¼ to ½ mile in width. This is the site of the reservoir formed by the dam built at the head of the gorge.

The construction of this dam was decided upon and work begun in November, 1886. The original plan designed was a narrow wall of concrete masonry, 50 feet high, 10 feet wide at bottom, 3 feet on top, arched upstream. On the upper side an embankment of loose earth was to be filled in against the masonry wall to its full height. After two months' work had developed the character of the design, the plan was disapproved by the management, and the writer was called upon to design a suitable structure and execute its construction. Some thirty-five thousand dollars had already been expended, and in order to utilize as much of the old work as possible, the new structure was planned to rest upon and encase the foundations already laid. This decision influenced to some extent the radius of the arch of the new dam, as well as its position relative to the axis of the cañon and the location of the anchorage on the sides. In other words, to avoid throwing away the work already done, the new work was adapted to the old in a way that ultimately increased the length of the dam on the crest somewhat more than would have been necessary by shifting the point of radius to one side of the central axis of the cañon, and making the radius somewhat shorter than it otherwise would have been. An engineer is sometimes driven to adaptations of this sort against his judgment, to save, or to give the appearance of saving, the pockets of his employers.

The modifications of the original plan were radical ones. The combination of earth and masonry was rejected, as it seemed to the writer that water alone was sufficiently heavy for the masonry wall to support without adding the last straw on the camel's back, of a mass of saturated earth. A gravity profile was adopted, and rubble masonry formed of blocks of stone up to four tons weight, was substituted for bastard concrete composed of cement mortar, with small stones rammed into it, which had been previously used. So much of the old plan was retained, however, as to form an embankment 50 feet wide on top, 10 to 15 feet high across the cañon, against the face of the wall, but clay, well rammed in layers, was substituted for the silt and quicksand loosely dumped, with which the dam was formerly being made. The object of

this clay-filling was to cut off possible seams in the bed rock underneath the dam, and reduce the pressure on the structure. The top of the embankment is 70 feet below the top of the dam.

The Foundation.—After the bowlders, sand and gravel had been stripped from the base of the dam on either side of the old work, the bed rock was found to be very irregular in surface, presenting the appearance of a number of pyramids and cones thrown heterogeneously together, but bound solidly in one mass, and well polished by attrition. The rock was very close in texture and exceedingly hard. No attempt was made to cut out the bed in level benches, as the unevenness of the bottom, as nature left it, gave the assurance that whatever movement might occur in the structure built on such a base, there could be no possibility of its slipping or sliding on the base. Wherever there were seams in the rock they were invariably occupied by roots, and the excavation was carried down till the seams pinched out and the roots disappeared. The rock was then thoroughly scrubbed by hand, and a thin grout of pure cement applied with brooms, filling the minutest crevices and angles in the rock, before starting the masonry.

The side walls of the cañon required more excavation to reach a satisfactory anchorage than the bottom. The north side was composed of shattered rock scored with innumerable seams, filled with red clay. In this material the excavation was carried to a depth (perpendicular to the slope) of 20 to 25 feet, before a solid ledge, free from seams, was encountered. This ledge lay with a slope nearly parallel with the surface slope, and in direction so nearly parallel to the radial line of the curve of the dam, that it could not have been better placed to receive the arch thrust, and formed a natural skewback. This was carefully stripped and treated with cement grout in the same manner as the base.

The abutment on the south side was against the end of a dyke of trap rock, crossing over the hills to the south in a direction nearly parallel to a line passing through the center of radius, and dipping westward at an angle of about 10 degrees from the vertical. After cutting into the face of this rock 5 to 10 feet, all seamy, loose material was stripped away, and a bedding that was deemed sufficiently good was obtained, although the rock was not as free from seams nor as solid in mass as the north abutment. However, the entire foundation is an admirable one, of rock in place throughout.

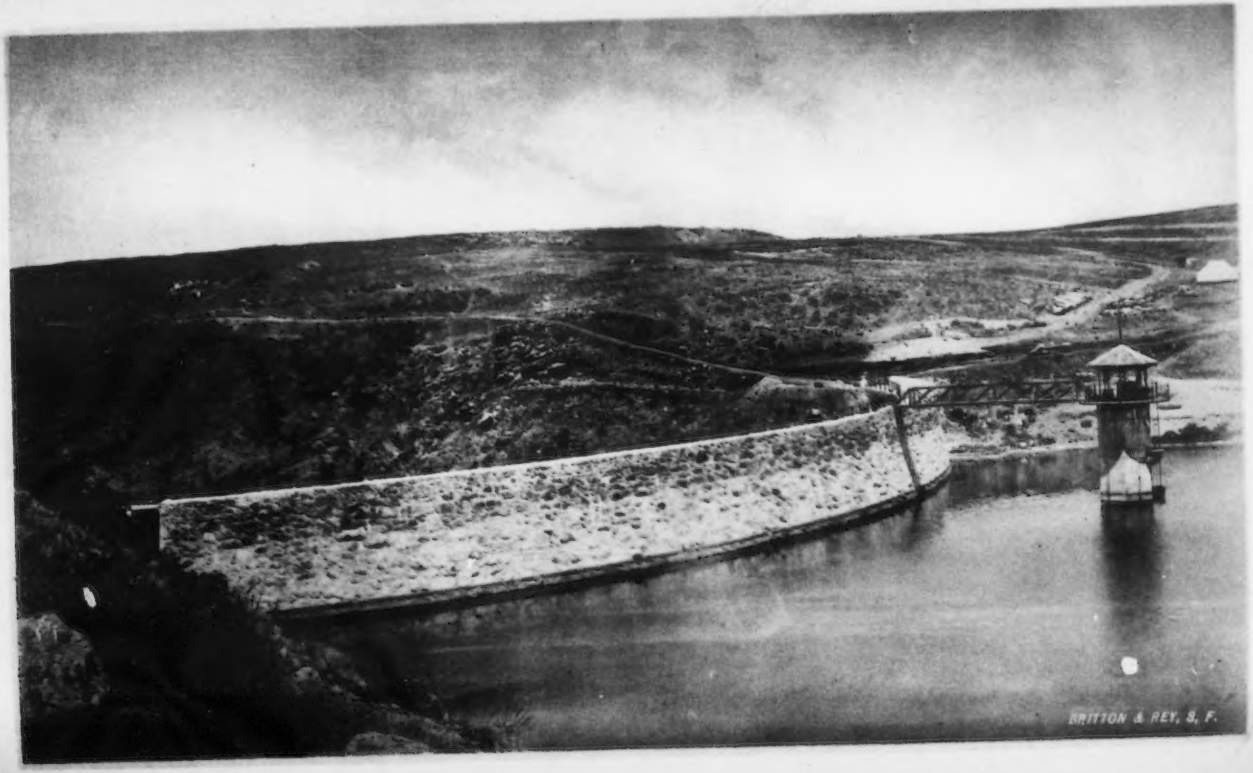


Plate XXIV.—Sweetwater Dam.

The Plan.—The original height of 50 feet was arbitrarily adopted at the beginning of the work, without any special investigation of the quantity of water to be stored by a dam of that height, but was "guessed" to be sufficient for present necessities, and the estimate of its cost was considered to be about the limit of the expenditure the company cared to make on an experimental scheme. There was an immediate and pressing need for water, the rainy season was passing, and it was desired to get up a part of the structure as rapidly as possible in order to catch a partial supply for the coming summer. Accordingly, in compliance with this desire, the foundation was rapidly laid and the structure hurriedly carried up to a point where it was safe to begin catchment. The base of the dam was laid with a width of 36 feet, and at a height of about 15 feet above the lowest course, it was drawn in to a thickness of 24 feet. At this level (whose elevation above tide is 140 feet) the lowest pipes pass through the dam. Above this level the structure was carried to a height of 45 feet, with a top width of 5 feet, base as stated of 24 feet, face batter (upstream) of 1 to 6. In anticipation of a probable addition to the height of the dam in future, the back was built in three steps, to give an opportunity of bonding the new work to the old. The profile of this portion of the structure is shown in Plate XXX. It was a gravity profile, whose line of pressure passed within the inner third of the base. It was constructed in arch form, convex to the stream, on a radius of 225 feet on the face line at top.

During construction the stream was carried in a conduit 30 inches square through the masonry near the bottom of the original creek bed. But one storm of the season of 1886–87 (a dry one) swelled the creek sufficiently to exceed the capacity of this conduit, and then it rose and ran over the top of the masonry for two days only, without injury. This occurred February 14th and 15th, 1887, when the flow reached a maximum of about 500 cubic feet per second. The gate at the upper end of the conduit was finally closed April 20th, 1887, and the conduit was filled solid with masonry from below. From that time until June 1st, the catchment was about 80 000 000 gallons.

By the 1st of June the structure, as planned, was completed to the height of 60 feet above the bottom, 10 feet higher than the height originally contemplated. It contained about 7 500 cubic yards of masonry and had cost all told (including the preliminary experiments) about

$100 000. Meantime, surveys of the reservoir basin and watershed had developed the fact that the 60-foot dam would impound 1 221 million gallons, whereas, its extension to 90 feet in height would give a capacity of nearly five times that quantity, or 5 882 million gallons. Also that the area of the watershed tributary to the dam is about 186 square miles, of which one-third is above an elevation of 3 000 feet, and between that elevation and 6 500 feet. The watershed was evidently ample to justify the hope that the greater reservoir would be filled almost every year of ordinary rainfall. The increased volume of water stored would so largely extend the utility of the works, and give so considerable increase in security against the disasters following a severe drouth, that the increased expense of extending the height of the dam while the working force and plant were on the ground and fully organized, seemed to be immediately justifiable. These arguments were embodied in a report, which was favorably considered by the directors of the company, and orders were given, about a fortnight before the 60-foot dam was completed, to extend the structure to a height of 90 feet.

This somewhat extended account of the growth of the enterprise from small beginnings is necessary to an understanding of the causes that led to the building of the structure in two sections rather than as a mass. The fact is that the work was nearly half done before all the conditions were thoroughly understood—conditions which ordinarily in works of such magnitude and importance are known, studied and exhaustively discussed preliminary to the beginning of any work whatever.

In designing the plan of the higher structure, greater reliance was placed upon the arched form than in the lower dam, then approaching completion. The profile adopted was one which theoretically gave stability by its own gravity, but without as large a factor of safety. The line of pressure falls about the center of the lower third. It was reasoned that as the foundation was as near perfection as can be generally found, apprehension on that score was unnecessary—and the source of the failure of such of the great masonry dams of the world as have given way—insecure foundations—need not be regarded as a factor in this case. If one can imagine a monolith to be carved in the form of a true arch, of such weight and dimension that any section of it is capable of withstanding the pressure of quiet water against it to its full height, without sliding or overturning, and such a monolith be firmly wedged

between the rock-bound walls of a narrow cañon, the possibility of its being ruptured, displaced or destroyed from water pressure alone, cannot readily be conceived. Now, if by the use of rich cement mortar and the best of building stone a structure be formed of the same dimensions and in the same position, which in time becomes virtually a monolith, based on the firmest of bed rock, its stability must be equally assuring to the mind.

The dimensions adopted were the following: base, 46 feet; top thickness, 12 feet; height, 90 feet; radius of arch, 222 feet on line of face at top. The face batter of 1 to 6 was carried to within 6 feet of the top; thence to the top of the parapet wall, vertical. The batter on the back started at the top with 1 to 3 for 28 feet; thence 1 to 4 for 32 feet; thence 1 to 6 to the coping.

The Construction.—When the new work was begun at the base of the completed structure, special care was taken to secure a perfect footing for the toe. When the foundation was stripped it was found that there was a slight leakage at various points along the bottom of the masonry, amounting altogether to about 10 000 gallons daily. The only perceptible leakage through the masonry was along the sides of the waste conduit, which had been recently filled in, although there were moist spots all along near the bottom.

All the leakage was entirely cut off by the new work, although it was necessary to carry up small well-holes alongside the old masonry to within about 15 feet of the level of the water surface in the reservoir, and keep them pumped out, before it was safe to close them entirely. Water was standing in the reservoir at a height of about 35 feet above the base of the dam, and the small quantity of leakage, and the ease with which it was stopped, was considered a favorable test of the superior quality of the masonry.

The stone used was of two grades, a dark blue and a gray metamorphic rock, impregnated with iron. The gray stone is full of minute quartz crystals, and is of slightly less specific gravity than the blue stone, which carries more iron. It was obtained from a quarry opened in the face of a vertical cliff over 100 feet high, situated 800 feet below the dam. It has no well-defined cleavage, and broke out in irregular masses, although generally having one or more tolerably smooth faces. Numerous tests of its specific gravity gave its weight as 175 to 200 pounds per cubic foot. The average weight of the masonry in place was

estimated at 164 pounds per cubic foot, which was the value used in the calculation of stability and strains.

Portland cement of the best obtainable quality was used in the proportion of one part of cement to three parts of sharp river sand. For the upper 4 feet next to the water a richer mixture of one to two was used. The sand was clean, sharp, and of the most suitable degree of coarseness to make the best of mortar.

In all the later portion of the work, from May 1st till its final completion, the mortar was mixed in a machine invented and patented by Mr. E. L. Ransome, of San Francisco, consisting of a cubical dice-box suspended on bearings attached at two corners diagonally opposite, through the center of which passed a perforated tube for injecting water, the box being revolved by horse-power. The ordinary charge was three barrels of sand and one barrel of cement, which was dumped into a hopper from a platform above the mixer, and admitted into the box through a door. The box was generally given three or four revolutions after charging with sand and cement before the water was admitted. A cock from a small tank regulated the flow of water, which was turned in slowly, the whole supply required being admitted in the next three or four revolutions. Eight to ten revolutions were sufficient to thoroughly mix the mortar, requiring two to three minutes in all. The batch, when mixed, was dumped into a box with a hinged bottom, resting on a low car, and run out on a tramway within reach of the derrick standing on the wall. Four chains of equal length, attached to the corners of the box with rings at the end, served to hoist it from the car, and the derrick delivered it where required on the wall, the latches holding the bottom in place were tripped, and the load instantly dumped within reach of a group of three or four masons.

After the completion of the 60-feet structure the tramway for delivering mortar was carried entirely around the face of the dam on a bracket trestle suspended from bolts driven into holes drilled in the wall, the track being on a level with the top. A grade of 3 feet in 40 at the end next the mixer was sufficient to give impetus to the car and carry it to the farther end of the dam, and a brake was attached to stop it where required. This tramway did excellent service, and remained in the same position until the dam was carried to its ultimate height, 30 feet above the level of the tramway. Prior to adopting this device the mortar had been mixed by hand, and delivered by hod carriers. With

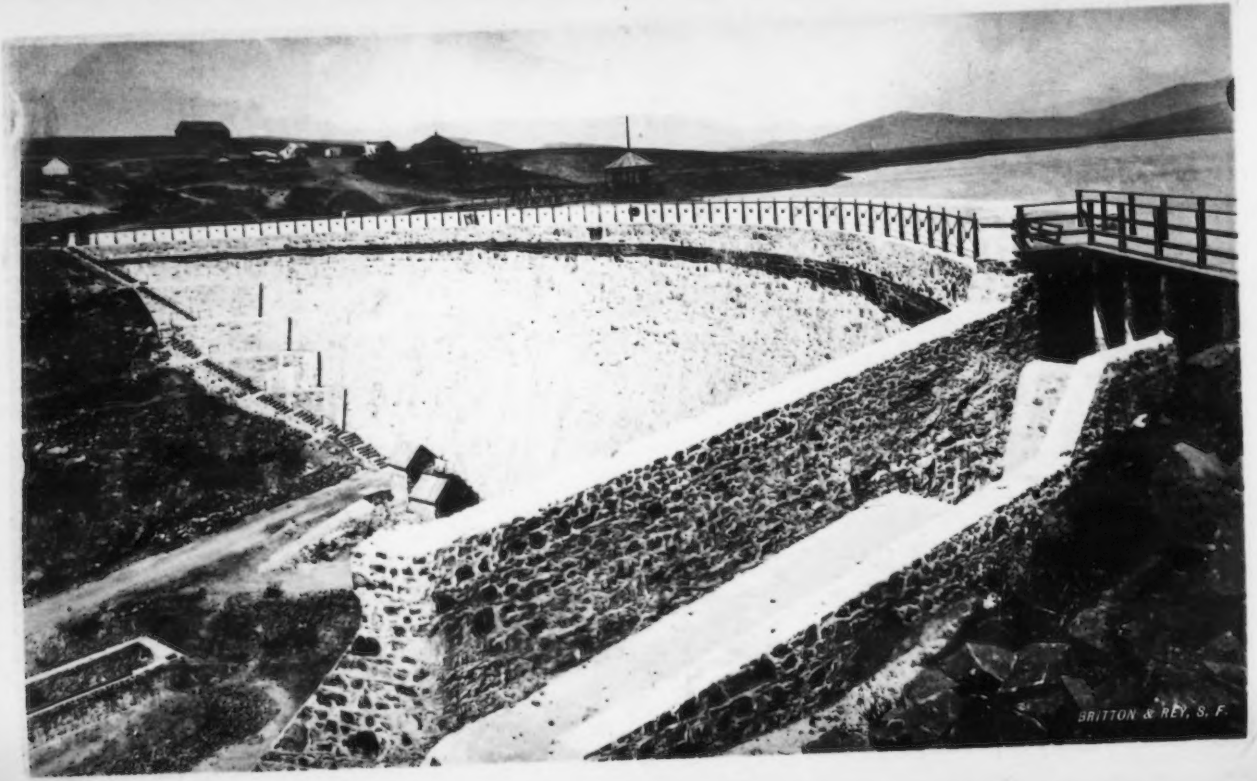


Plate XXV.—Sweetwater Dam.

the mixer and tramway, five men and a horse performed the work formerly requiring sixteen to twenty men (four mixers and twelve to sixteen hod-carriers), more promptly and satisfactorily. The hoisting of the mortar by the derricks was performed without interference with their regular work.

The stone was hauled on wagons, rigged up with platforms on a level with the hind wheels. The derricks in the quarry were simple shear-legs, slightly inclined from the vertical. The stone was hoisted under the center of the shear-legs, the wagon driven underneath it, the load gently lowered, and the chain left in position for use in hoisting at the wall. Stones weighing 6 000 pounds were not infrequent—and even greater ones were readily handled on the wagons and hoisted into position. All stones smaller than 500 pounds weight, were loaded on stone-boats, 4 feet square, made of 3-inch plank with a bottom of boiler plate. These were provided with chains at the corners for hoisting. They cost $30 each; were made very strong, and yet the wear and tear was so great that several sets of them were worn out on the job.

All the hoisting was done with horse-power. This was frequently criticised as questionable economy, but taking into consideration the scarcity and high price of fuel, the cost of the number of hoisting engines that would have been required, the delays occasioned by breakages, the skilled labor required to do the work, etc., the advantage in cost and convenience was on the side of animal power. The writer had a previous experience during the construction of a section of the San Francisco sea-wall, which led to this preference. Steam and horse-power were there applied to the derricks in the quarry, side by side, in the handling of over 100 000 tons of rock, and the greater economy and convenience of horse-power was marked and decided.

Four derricks were in constant use on the wall, and at one time five. The masts of four of them were 30 to 38 feet in length, the booms 26 to 32 feet. A larger one with mast 50 feet, and boom 45 feet, was so superior in efficiency to the others, that it was a matter of regret that all had not been of larger size. All the work was done without serious accident or loss of life, although there were narrow escapes, and several horses were killed or injured. It was doubtless due to constant vigilance and inspection of apparatus, that no fatal casualties were recorded.

The work was completed April 7th, 1888, having occupied sixteen months in construction, including two months of delay on account of shortage of cement.

The total volume of masonry laid was as follows:

In the dam proper	19 269.0	cubic yards.
" wasteway	481.2	"
" inlet tower	376.8	"
" conduit from dam to tower	182.0	"
" gate houses	71.0	"
In various accessories	127.0	"
Total	20 507.0	"

In this work 17 562 barrels of cement were used, an average of 1.17 cubic yards of masonry per barrel of cement.

The total cost of the dam was $234 074 11, distributed as follows:

PLANT.—Tools, etc			$6 236 76
MATERIALS.—Cement	$63 111 03		
Cement hauling	8 614 18		
Lumber	2 408 08		
Iron work	4 915 99		
Pipes, gates, etc	5 152 58		
Miscellaneous materials, powder, etc	3 229 84		
			87 431 70
LABOR.—Common and skilled labor	$93 590 55		
Foremen	6 866 49		
Teams	19 696 12		
Engineering, salaries and expenses	10 555 20		
Clerical work	653 88		
Earth work (contract)	7 666 51		
Miscellaneous expenses	1 376 90		
			140 405 65
Total			$234 074 11

The cost of the flowage tract for the reservoir is not included in the above. A little over one-half the land cost $16 426 93. The remainder is in litigation under an action of condemnation. A San Diego jury under the stimulus of "boom" prices awarded the owner $280 an acre, or a little over $100 000 for land, one-third of which was worthless, and the remainder unimproved. This judgment is being contested before the Supreme Court. The clearing and grubbing of about three hundred acres of the reservoir basin cost $10 808 46.

Measurements were taken weekly of the volume of masonry laid, and these measurements compared with the expense account, as a check upon the cost of each department of the work, and the system thus maintained throughout served to point out reforms needed from time to time. The average cost per cubic yard of 11 322 yards of masonry laid during the period from May 1st to December 31st, 1887, is shown in the following table:

Items.	Cost.	Cost per Yard of Masonry.	Percentage of Total Cost.	Remarks.
Cement—9 237 barrels at $4.20....	$38 795 40	$3.427	38.900	Average 1.22 per bbl.
Stone—Quarrying.....	4 816 23	.425	4.829	
Loading.....	5 917 83	.523	5.933	
Hauling.....	4 756 75	.420	4.768	
Hoisting.....	6 531 52	.577	6.550	
Sand—Loading and hauling.....	3 902 61	.345	3.915	
Mortar—Mixing and delivering...	2 702 27	.239	2.710	
Masons.....	9 025 05	.797	9.050	
Helpers.....	2 101 30	.186	2.109	
Excavations of foundations.....	3 433 66	.303	3.444	
Making and repairing roads.....	1 332 30	.118	1.336	
Blacksmithing (labor).....	1 847 97	.163	1.854	
Carpentry.....	1 100 29	.097	1.104	
Rope.....	1 181 70	.104	1.186	
Tools.....	521 68	.046	.524	
Steel.....	153 59	.014	.155	
Blacksmith coal.....	107 48	.009	.109	
Blocks and sheaves.....	131 44	.011	.133	
Powder.....	971 00	.086	.974	
Foremen.....	3 763 11	.332	3.774	
Engineering and superintendence	3 880 00	.343	3.891	
Lumber.....	2 212 92	.195	2.220	
Wetting masonry.....	540 15	.048	0.542	
Total.....	$99 726 25	$8.808	100.000	

The Wasteway.—This important adjunct to the dam was carefully considered and proportioned to carry the probable maximum flow of the stream that may be presented for discharge, with a full reservoir. It is located at the south end of the structure, and is 40 feet in length by 5 feet in depth, divided into eight bays of 5 feet each. These bays are formed by piers of masonry, set at right angles to the flow, and provided with recesses on the upper face, in which loose flash-boards of 2-inch plank rest on an incline of 35 degrees from the vertical. Any set of boards may be removed from top to bottom, or the water may be held at successive levels from the top to the bottom of the weir by removing the top boards all the way across. The water falling over the weir drops

into a series of pools, 3 feet deep, which relieve the structure of shock, and passes down an inclined plane with a fall of 1 to 10, until it is carried away from the dam a distance of 50 feet, and then plunges into the cañon below. The capacity of the wasteway is about 1 500 cubic feet per second. This may be increased to about 1 800 cubic feet per second by opening a 30-inch blow-off gate in the main pipe below the dam.

The Inlet Tower.—This structure is located 50 feet above the dam, nearly opposite its center. It is built of masonry, with cement mortar mixed two to one, plastered outside and in with two coats of mortar mixed with one of sand to one of cement. It is 16 feet square at the base for a height of 10 feet, where its form is changed to a hexagon, with walls of a uniform thickness of 3 feet to the top. Each of the sides of the hexagon measure 3 feet on the interior face. Into the walls of the tower are built seven cast-iron elbows, at an elevation of 10 feet apart from bottom to top, the upper one being 10 feet below high-water line. The bell-mouths of the elbows open upward, and are ordinarily closed with a plain valve or cover of iron. The design is to draw water from the surface at whatever stage it may be. When any one cover is removed, a basket screen is lowered in its place, fitting closely into the mouth of the elbow. Three pipes pass through the dam and enter the tower at the bottom. The two lowest pipes are of cast iron, 14 and 18 inches in diameter respectively, and lie side by side. They are encased in concrete throughout, from the tower to the dam. On top of them is built a conduit of masonry with a circular orifice 40 inches in diameter, formed of walls 30 inches in diameter, in double arches. This conduit leads from the interior of the tower to the center of the dam, where it joins a pipe of ½-inch boiler-iron, 36 inches in diameter, leading to the main gate immediately below the dam, and from this gate is carried the main pipe line down the valley. The smaller pipes are not at present used, except to supply a hydraulic ram throwing water to the keeper's house on the hill, 150 feet above, and to drain the tower when all the valves are shut. They are intended to be used for supplying a turbine and pump to throw water to a higher level than the dam will now reach.

As an illustration of the fact that masonry laid in Portland cement in the proportion of two of sand to one of cement, may be made water-tight with care exercised in laying, this tower, and the conduit leading from it, may be cited. When they are empty the pressure from the outside at present is somewhat more than 20 pounds per square inch on the

Plate XXVI.—Sweetwater Dam.

conduit and at the bottom of the tower, and there is no leakage in either of them.

The Reservoir.—Red clay soil constitutes the bed of the reservoir basin, or the major portion, outside of the old river-bed and bottoms, and is of an impervious nature. The following table of area and contents of reservoir is presented:

Contour elevation.	Area in acres.	Contents. Gallons.
145 feet (level of lowest outlet valve in tower)	3.51	
150 "	10.72	11 640 000
155 "	17.12	30 577 000
160 "	43.10	79 631 000
165 "	75.21	175 819 000
170 "	113.40	329 546 000
175 "	153.75	547 069 000
180 "	200.77	835 851 000
185 "	272.22	1 221 355 000
190 "	326.96	1 710 583 000
195 "	397.85	2 302 261 000
200 "	463.80	3 005 642 000
205 "	538.94	3 824 197 000
210 "	630.94	4 778 549 000
215 "	721.86	5 882 278 000

The table shows that eighty per cent. of the capacity of the reservoir is within the upper 30 feet of height, and that forty per cent. is within the last 10 feet. This fact reduces within small limits the fluctuation of head on the mains after the reservoir is once filled, and constitutes one of the reasons for increasing the height of the structure, as it enables the establishment of the probable limit of irrigation on the lands below, at a line not lower than 25 to 35 feet from the top of the dam. The irrigable area was thus largely increased, by reason of the decrease in fluctuation of depth in the reservoir.

A line has been drawn upon the map, Plate XXXI, which represents approximately the limits of the irrigable land directly commanded by the reservoir.

The cement used in the construction of the dam was principally such as could be obtained in the market of San Francisco, at a time of extreme scarcity. White's, Gillingham, Knight, Bevan & Sturge and Kaufmann & Lieffmann (German cement) were the brands chiefly used; but at times it was necessary to take inferior brands, and reduce the proportion of sand, to keep the work going. The average cost of the

cement laid down in San Diego was $3.66½ per barrel; seventy cents per barrel was the price paid for hauling the greater portion from San Diego to the dam. Just as the work was nearing completion, a branch of the National City and Otaz Railway was finished to the dam, and a part of the cement delivered by that means at less rates.

Price of Labor.—Masons were paid $4 to $5 per day; common labor, $2 to $2.50; foremen, $4 to $6; carpenters, $3.50 to $4; blacksmiths, $4; teams, including driver, $5; machinists, eighty cents to $1 per hour. The work was done in the midst of the "boom" in Southern California, when labor of all kinds was difficult to obtain, independent and restless on account of the general feverish excitement, and inclined to pick up at any moment and move on in search of better pay. All classes of supplies, tools and materials were correspondingly higher than the ordinary prices. These conditions increased the cost of the work twenty to twenty-five per cent. above the normal.

There has been no lack of wiseacres who predicted the failure of the enterprise as a permanent irrigation scheme; and some of the most intelligent citizens of the country criticised the location of the reservoir so near the mouth of the stream, on account of its presumed liability to obliteration as a reservoir by reason of the deposit of sand and silt. A careful examination of the water of the stream at flood time, when it was most heavily charged with sediment, convinced the writer that fears of this nature were groundless. An estimate made, within reasonable limits, indicated one thousand years as the time when it might be expected to fill. Samples taken by the State Engineer Department of California of the water of the Yuba, Bear, and American Rivers, immediately below the hydraulic mines, yielded an average of only about one half of one per cent. of sediment. Were the Sweetwater as heavily charged, it might fill the reservoir basin in one to two hundred years, but the voids would still contain a considerable volume of water that would drain out and be available, and the utility of the reservoir would not be destroyed if it were entirely filled with sand.

The Distributing System.—From the dam to the lower end of the cañon, 1 600 feet, the main pipe is 36 inches in diameter, and covered with masonry laid in lime mortar, plastered with cement. From this point it is reduced to 30 inches diameter, and follows the valley for 5 miles, and thence rises to the top of the Chula Vista Mesa 92 feet above sea level. Its entire length is 29 800 feet, and at its terminus the water

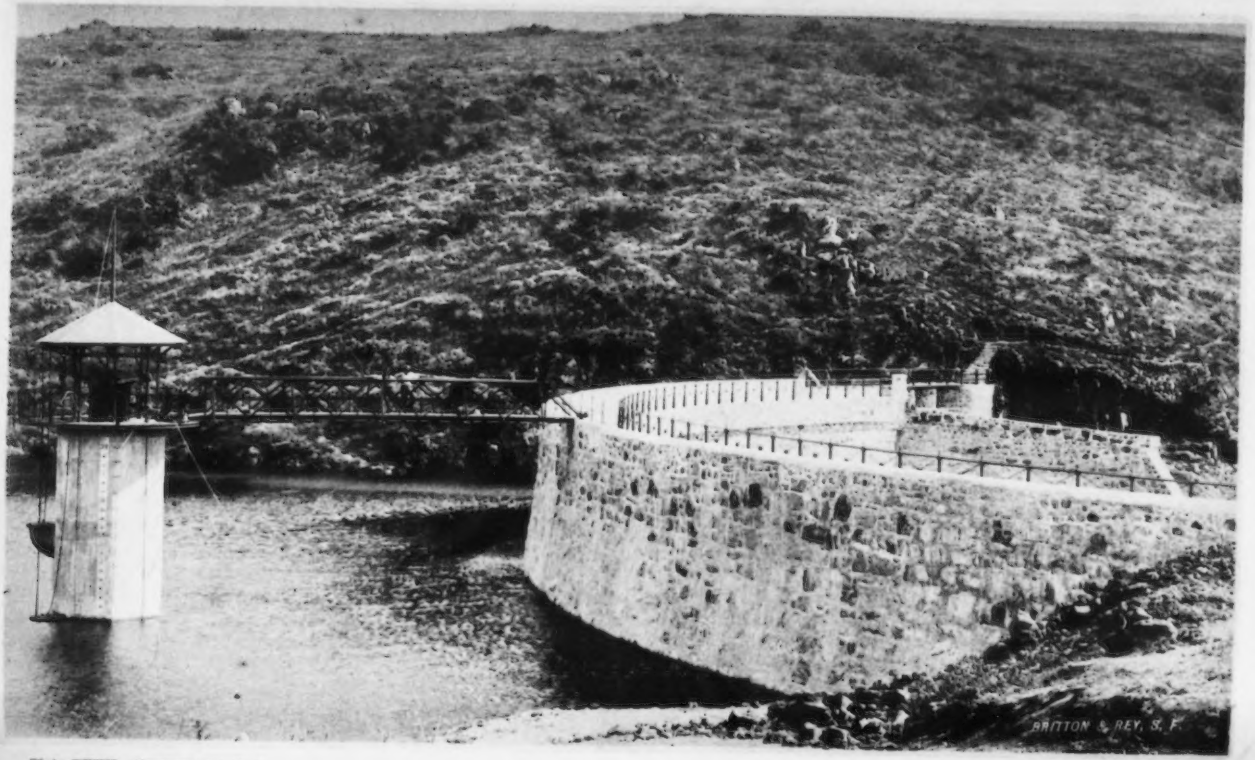


Plate XXVII.—Sweetwater Dam.

is divided into two 24-inch pipes, one running south 1 mile, the other west half a mile, where it is reduced to 18 inches diameter, and is carried northward to and through National City.

At the terminus of the 36-inch main a blow-off gate is located, to be used as a relief to the wasteway of the dam in case of a sudden flood which might exceed the capacity of the wasteway, or to draw off the water from the reservoir if, for any cause, it was desired to do so.

Wrought-iron pipes were used throughout. The total length of mains and laterals that have been laid is 58 miles, with 5½ miles on hand to be laid this season. They are of three classes, viz., straight double riveted pipe, manufactured and laid by the Risdon Iron Works, San Francisco; converse lock joint, kalamined lap-welded tube, made by the National Tube Works of McKeesport, Pa.; and spiral riveted pipe made by the Abendroth Root Manufacturing Company, New York. About 16 per cent. of the pipe was of the first class, 72 per cent. of the second, and 12 per cent. of the third. The length and diameter of each class furnished was as follows :

RISDON IRON WORKS, SAN FRANCISCO.

Wrought-iron, straight riveted.....	36 inches diameter..	1 594	feet.
	30 " ..	28 213	"
	24 " ..	2 034	"
	18 " ..	16 468	"

NATIONAL TUBE WORKS.

Kalamined tube.....	12 inches diameter..	25 903	feet.
	8 " ..	7 620	"
	6 " ..	132 333	"
	4 " ..	50 745	"

ABENDROTH, ROOT & COMPANY.

Spiral steel and iron.....	24 inches diameter..	5 950	feet.
	12 " ..	10 029	"
	8 " ..	4 020	"
	6 " ..	17 870	"
Total.....	.....	302 779	"

The introduction of spiral pipe into the system was unfortunate, as it does not stand the test of transportation across the continent, and will

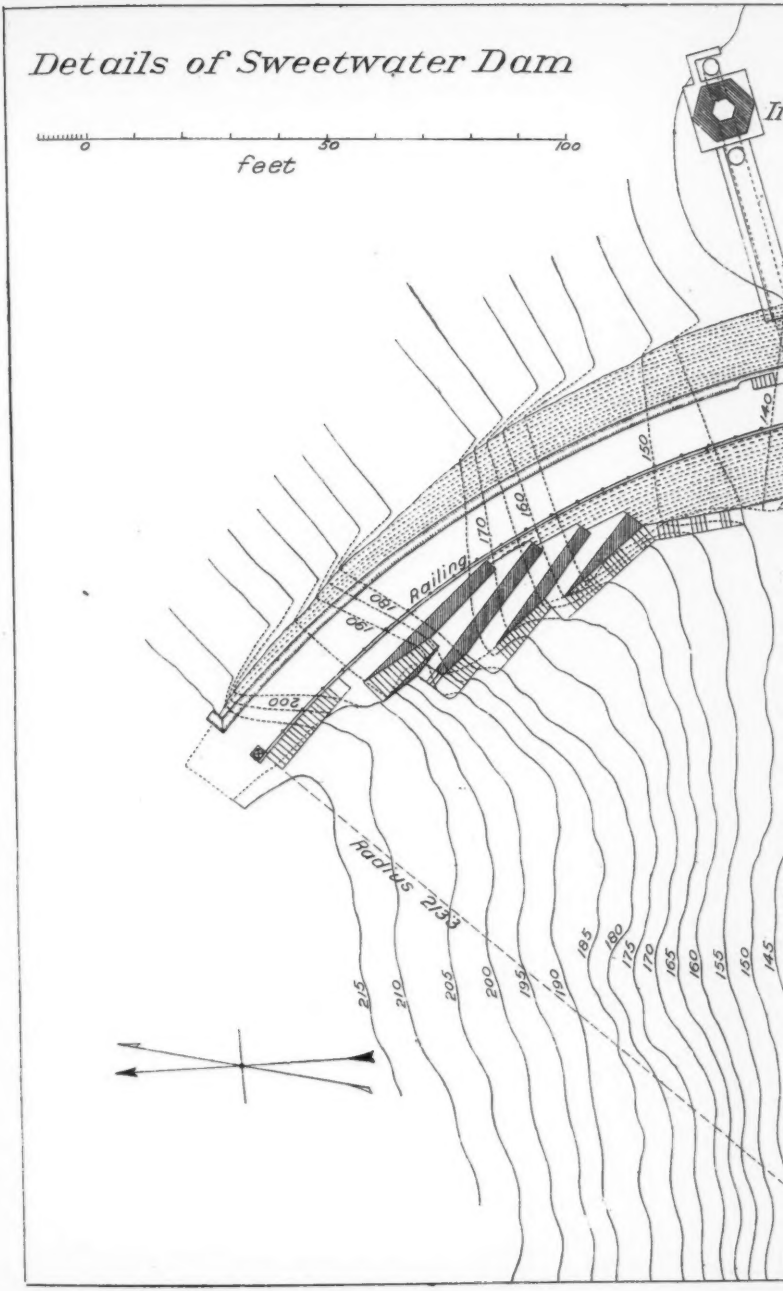
have to be taken up and specially treated to make it water tight. It will answer very well for sub-irrigation, if it could be properly controlled, but as it is laid in streets and avenues that system is not desirable or conducive to comfort in traveling.

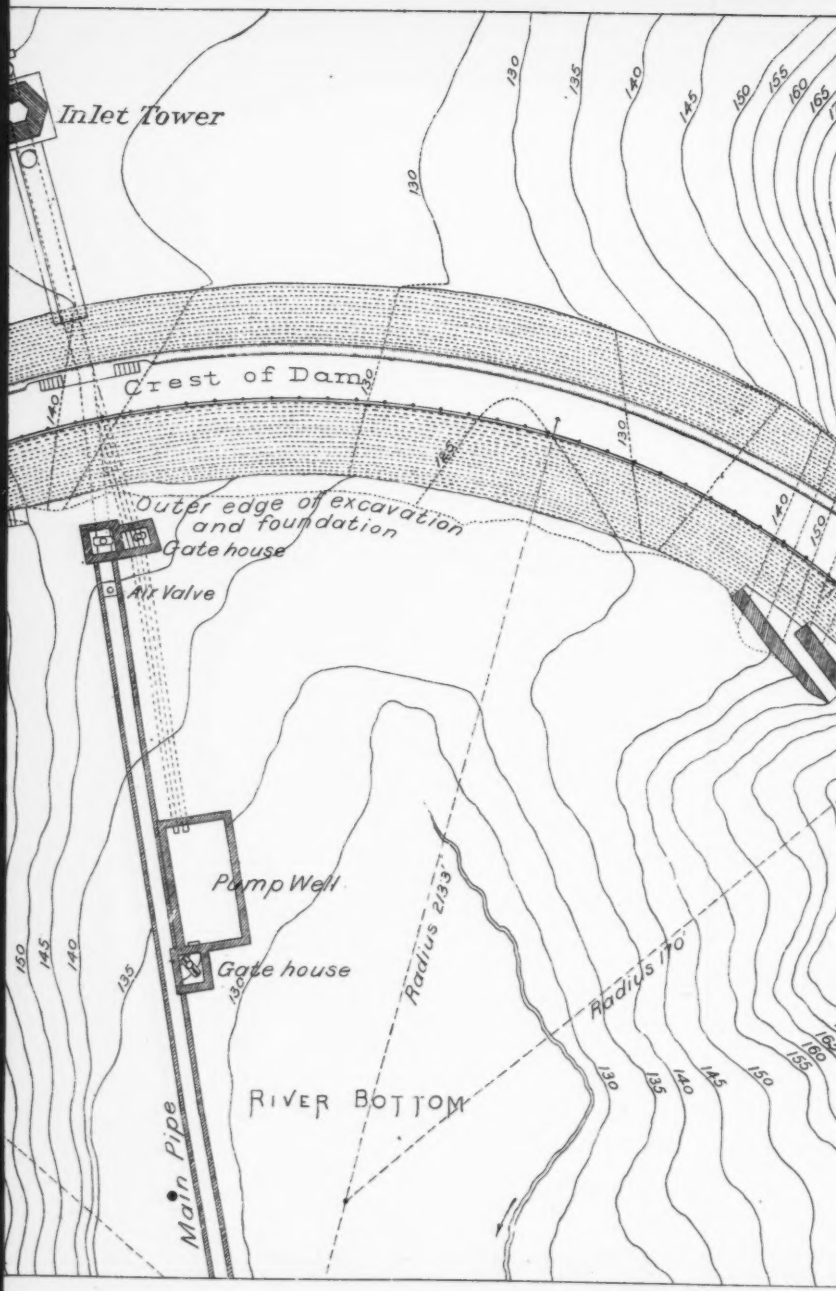
The total cost of the pipe lines was as follows:

Pipe	$301 928 80
Freight	39 183 03
Distribution	6 271 06
Gates	1 849 62
Materials, tools, etc	5 932 57
Right of way and miscellaneous expenses	2 968 00
Pipe laying	144 630 78
Total	$502 763 86

Probable Duty of the Works.—One of the most interesting questions to the Stockholders of the Company is the result that may be reasonably expected in the way of irrigation from such a reservoir. The assumption is made that in average years, say three out of five, the watershed will yield a sufficient supply to fill the reservoir, besides maintaining the consumption through the rainy season, thus starting on the irrigation season about May 1st with a full reservoir. From May 1st to October 1st is the average season of irrigation—about one hundred and fifty days. Where pipe distribution is in use, a fair average allowance in Southern California is a duty of ten acres per miner's inch (five hundred acres per cubic foot per second). There are instances of a much higher duty having been attained—a duty of even forty acres per miners' inch having been accomplished in one place. Allotting 700 000 000 gallons for the annual consumption of National City, and for loss by evaporation during the summer months, the remainder would yield a flow of 2 000 miners' inches per day for two hundred days; with a duty of ten acres per inch, this amount would irrigate 20 000 acres. In the course of time it is expected that a duty as high as twenty acres per inch will be reached, in which event, a reservoir full may be extended over two years' time, and still irrigate 20 000 acres, and afford a domestic supply to the town of National City.

Water rights, giving to the purchaser simply the privilege of becoming a customer for water have been sold on the San Diego Flume Company flume at the rate of $2 000 per miners' inch. At this rate the value


Details of Sweetwater Dam
0
50
100
feet
Railing
Radius 2133
140
150
160
170
180
190
200
205
210
215
195
185
175
165
155
145


Inlet Tower
Crest of Dam
Outer edge of excavation and foundation
Gate house
Air Valve
Pump Well
Gate house
Main Pipe
RIVER BOTTOM
Radius 233'
Radius 170
130
135
140
145
150
155
160
165

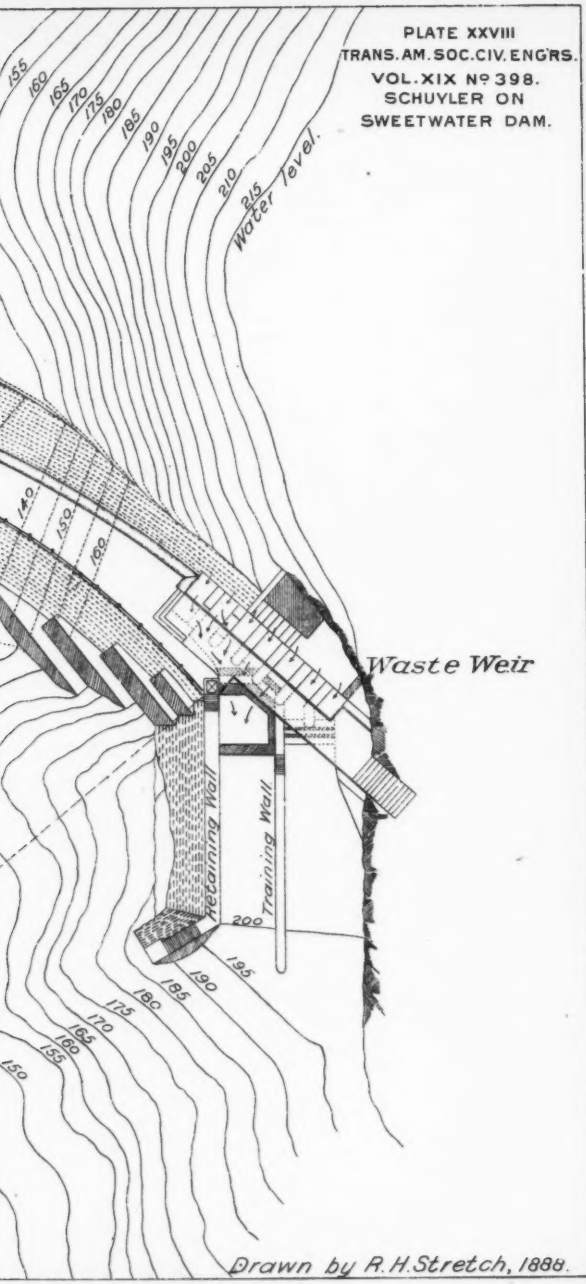


PLATE XXVIII
TRANS. AM. SOC. CIV. ENGRS.
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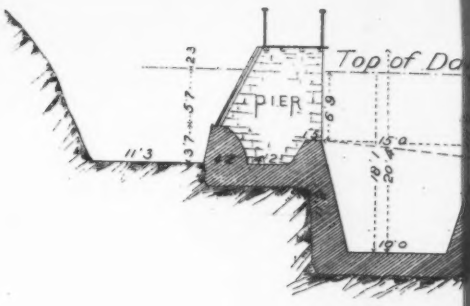
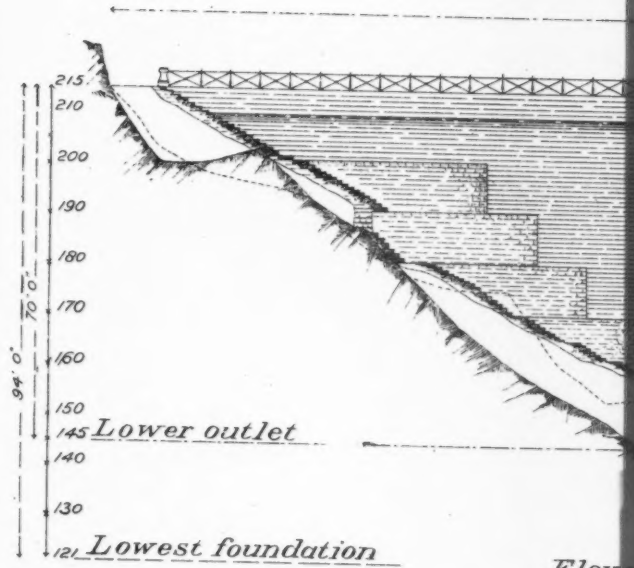
SCHUYLER ON
SWEETWATER DAM.
155
160
165
170
175
180
185
190
195
200
205
210
215
Water level.
140
150
160
Waste Weir
Retaining Wall
Training Wall
200
195
190
185
180
175
170
165
160
155
150
Drawn by R. H. Stretch, 1888.



Section of W

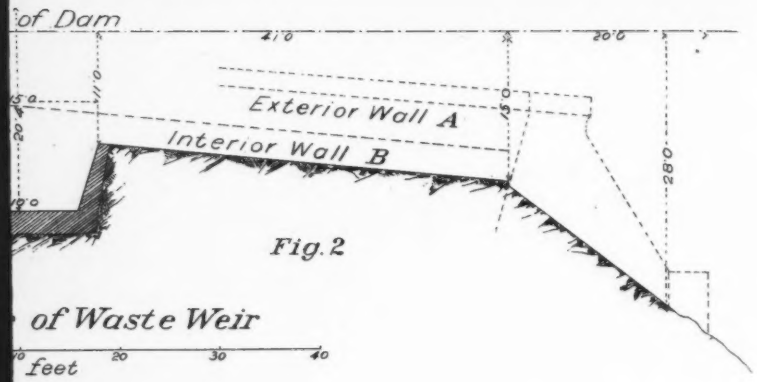
0 10 feet

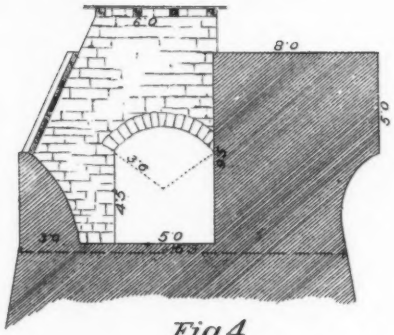


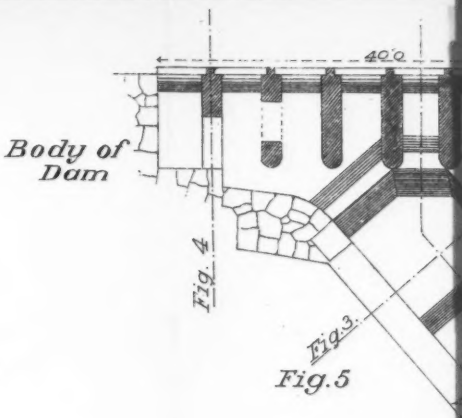
Elev

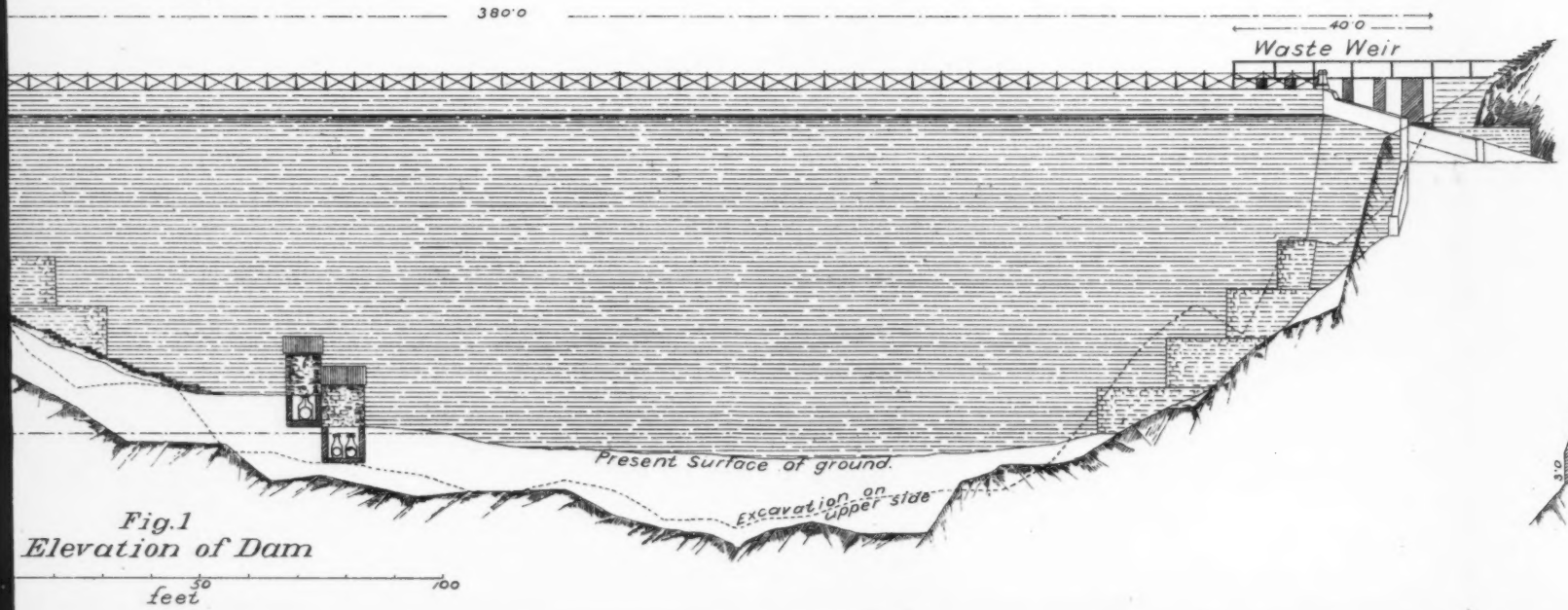
0 10

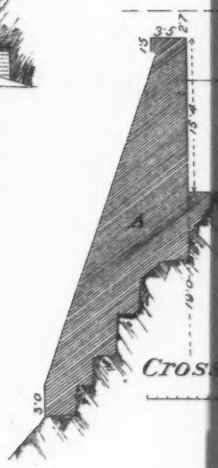
Details of Sweetwater Dam



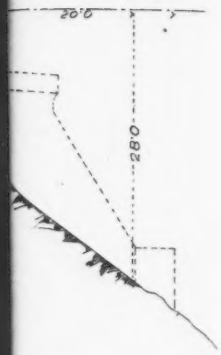








Details of Sweetwater Dam



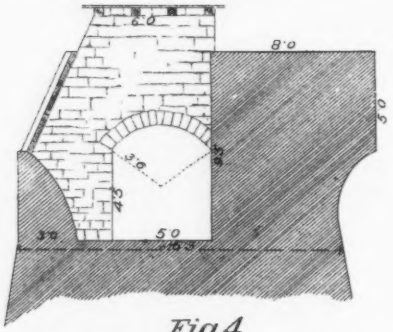


Fig 4
Section of Waste Weir
0 10 20
feet

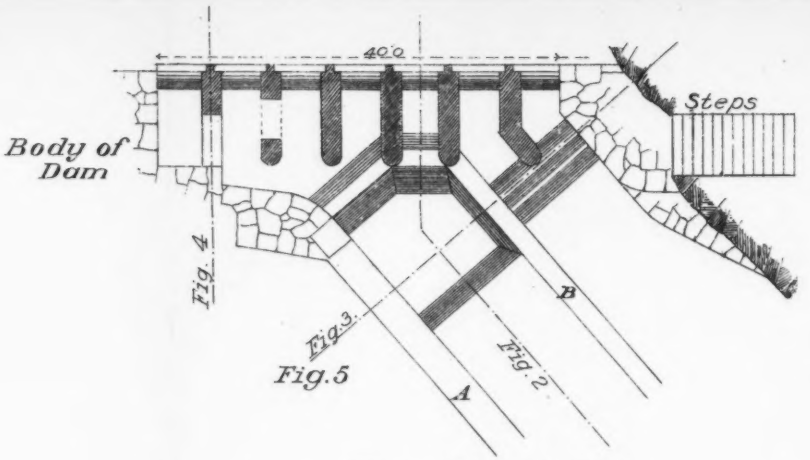


Fig. 5
Plan of Waste Weir
to locate sections.

PLATE XXIX
TRANS. AM. SOC. CIV. ENGRS.
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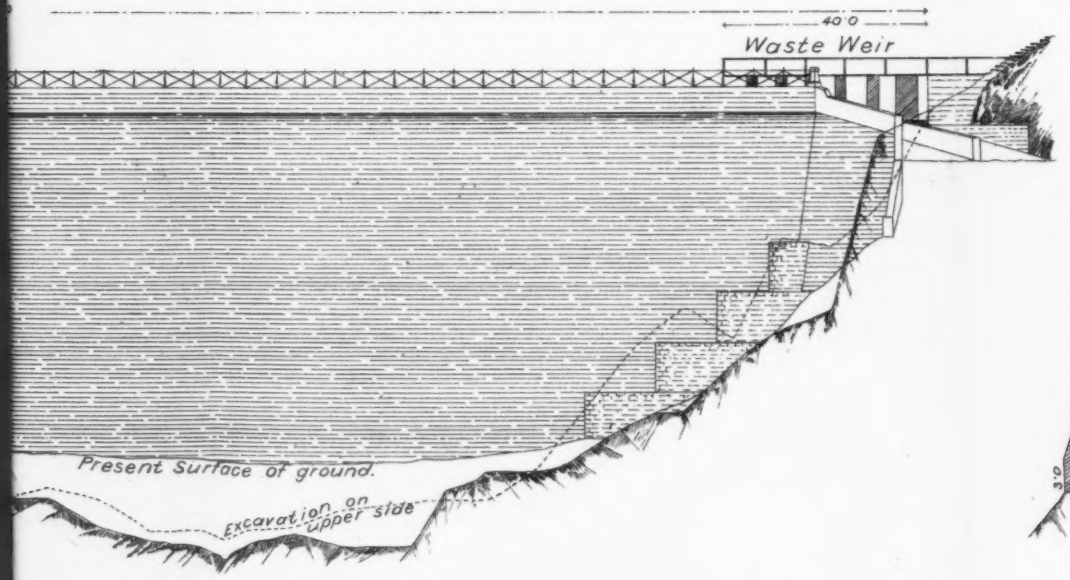
SCHUYLER ON
SWEETWATER DAM.



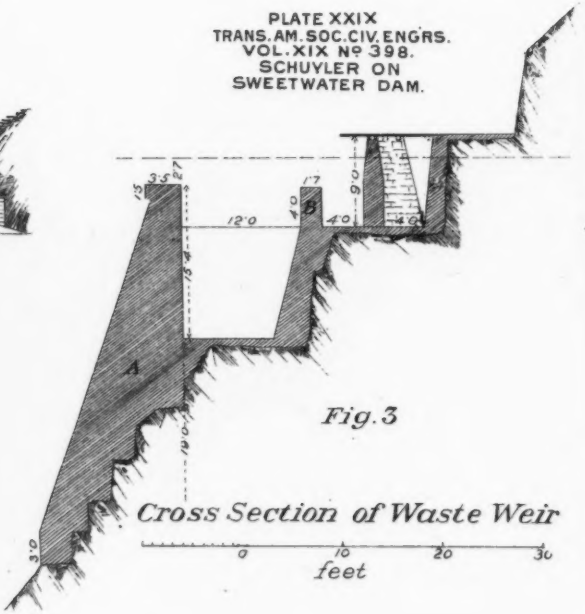


Fig. 3
Cross Section of Waste Weir

Drawn by R. H. Stretch, June 1888.

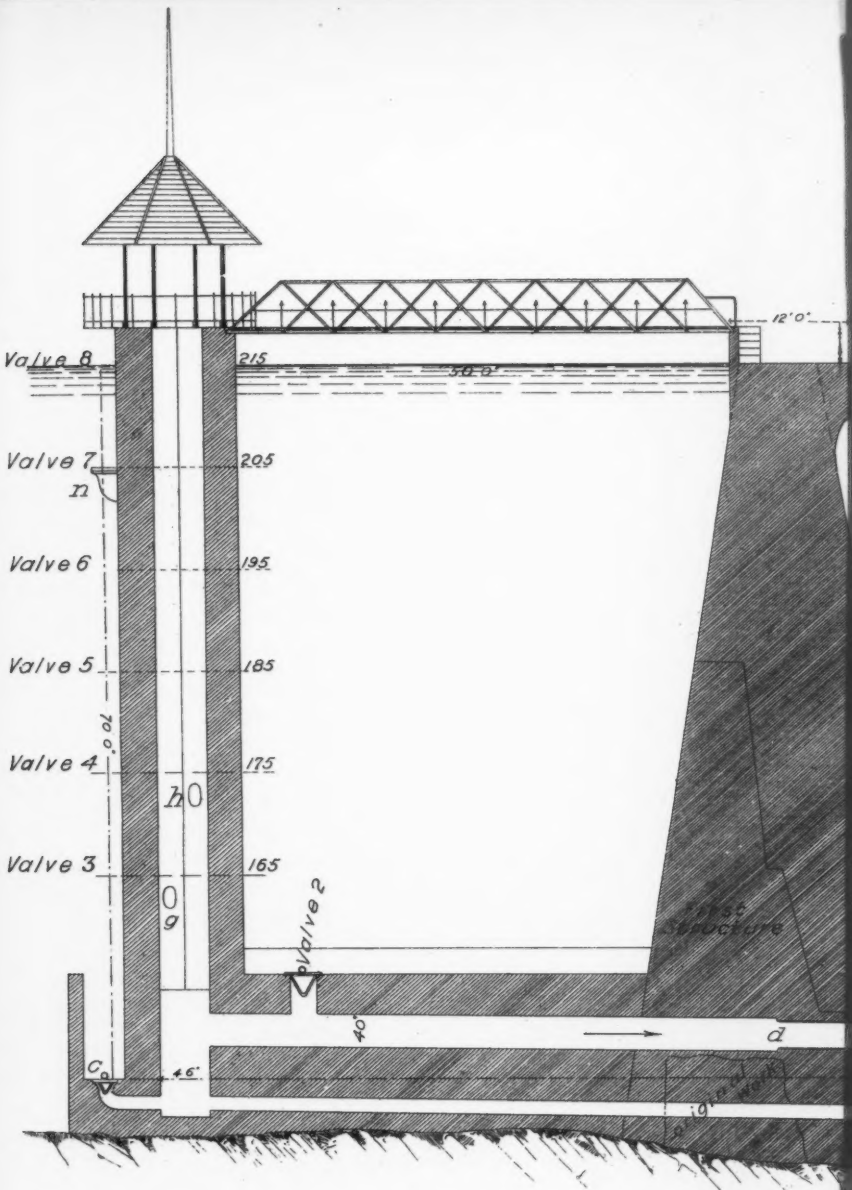

Valve 8
215
50·0'
12'0"
Valve 7
205
n
Valve 6
195
Valve 5
185
70·0'
Valve 4
175
h
0
Valve 3
165
0
g
Valve 2
40'
d
C
46'
Section of Dam

Fig.1

Details of Sweetwater Dam

PLATE XXX
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SWEETWATER DAM.

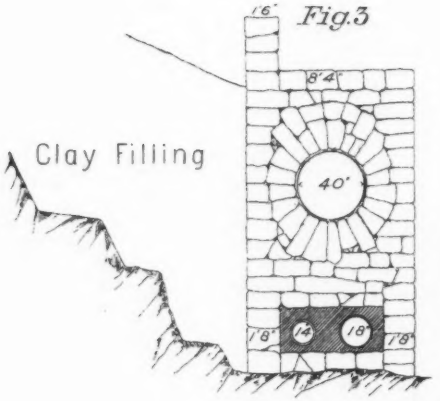


Fig.3

Section of conduit

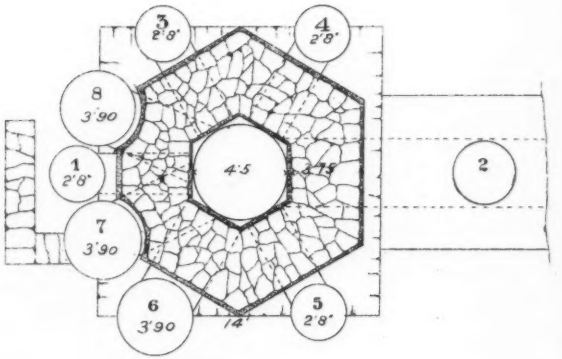


Fig.2

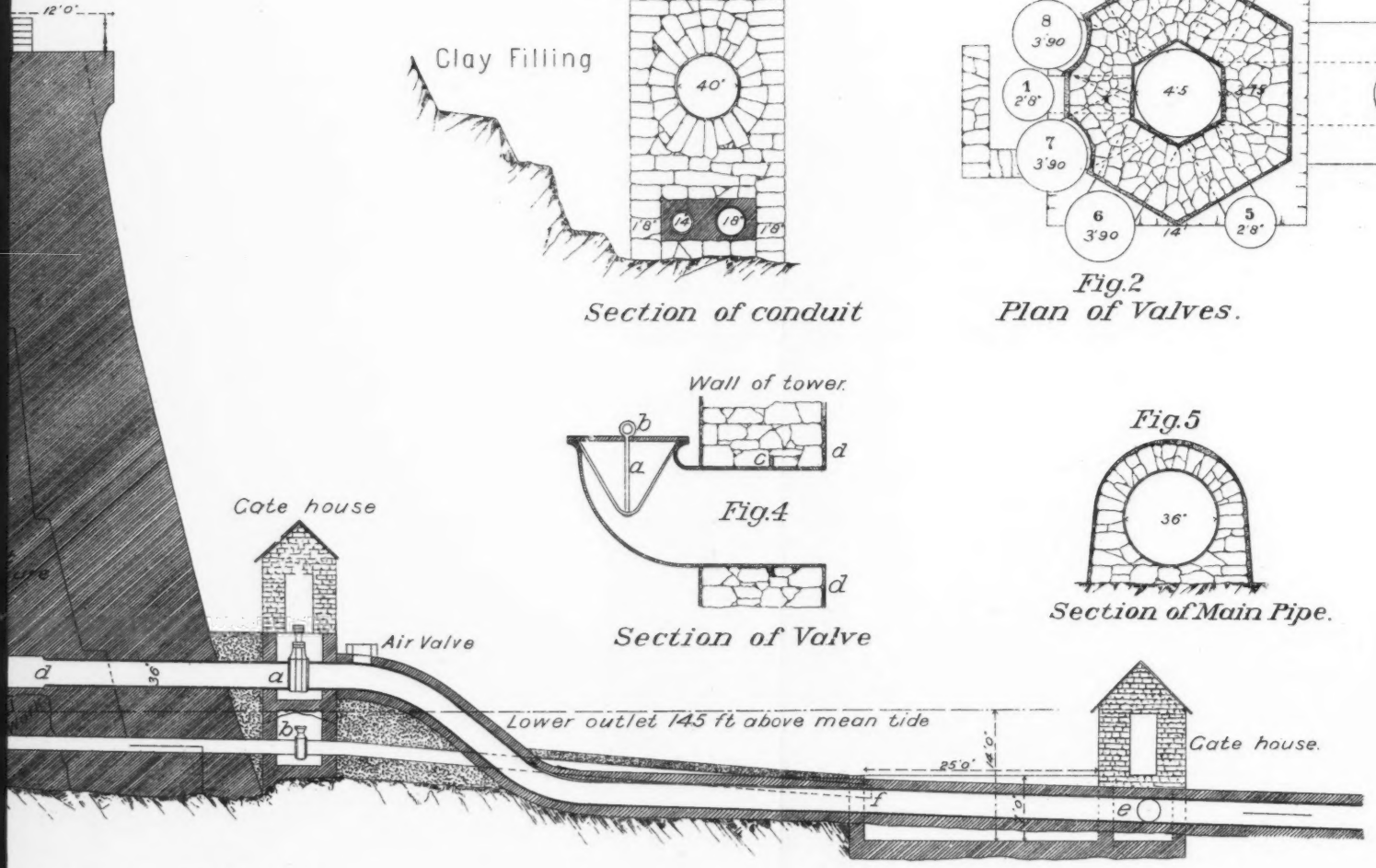
Plan of Valves.

Fig.4

Section of Valve

Fig.5

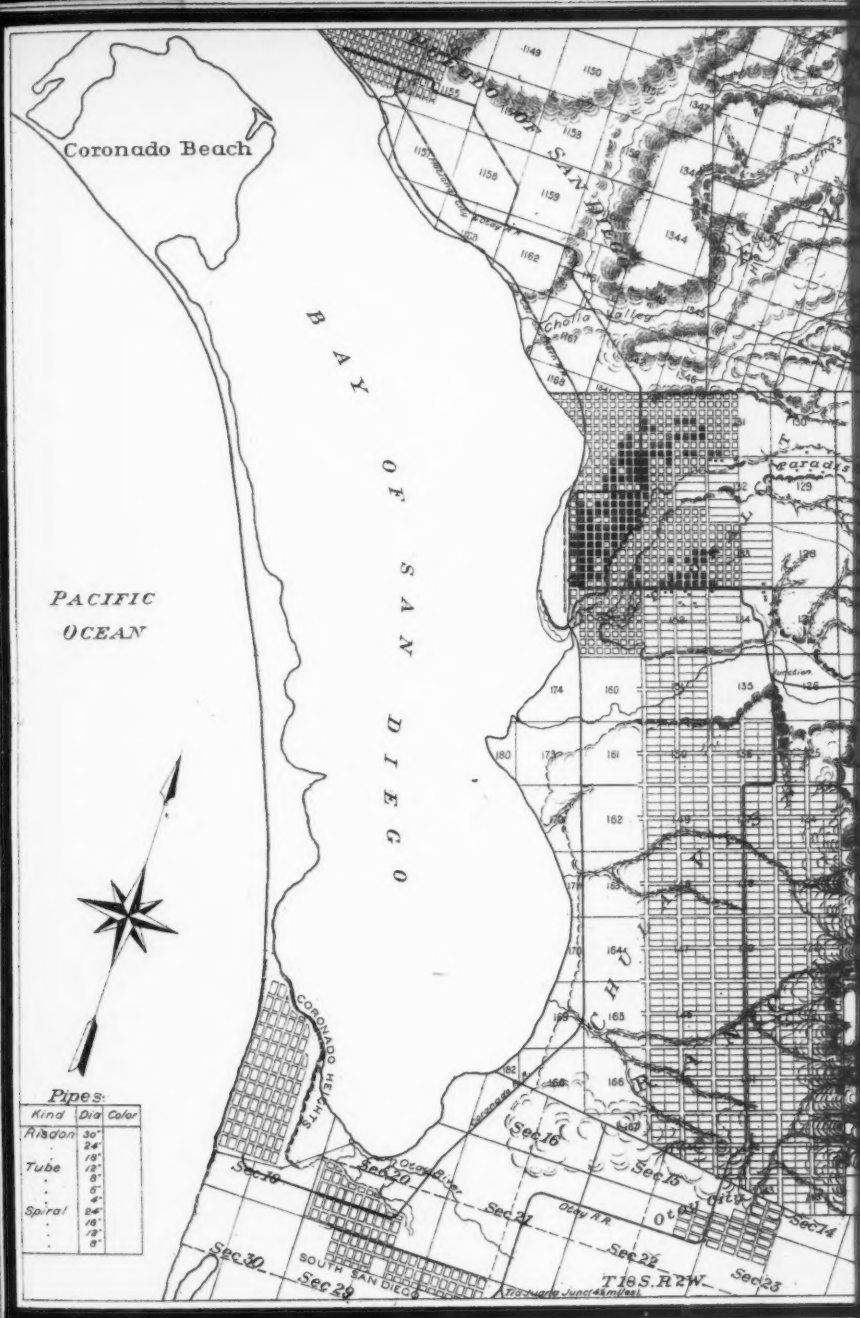
Section of Main Pipe.



of Dam and Outlets.

20 feet 30 40 50

Drawn by R.H.Stretch, June, 1888.


Coronado Beach
BAY OF SAN DIEGO
PACIFIC OCEAN
Pipes:
Kind
Dia
Color
Riveted
Tube
Spiral
CORONADO HEIGHTS
SOUTH SAN DIEGO
Sec 19
Sec 20
Sec 21
Sec 22
Sec 23
Sec 24
Sec 25
Sec 30
T 18 S. R 2 W.

MAP OF THE
SAN DIEGO BAY
REGION
showing the location
of the
SWEETWATER DAM
&
RESERVOIR
and the District commanded by
Pipe Lines.
SCALE OF MILES
MAY. 1888.
Sec 29
Sec 28
Sec 32
Sec 33
Sec 21
Sec 22
Sec 23
Sec 28
Sec 4
Sec 14
Sec 13
Sec 18
T17S. R1W
T18S. R1W.
Sweetwater Reservoir
Spring Valley
Sweetwater Valley
Lower Sweetwater Valley
Paradise Valley
Benson
Root
Jordan
Root
San Miguel Mt.
JANAL RANCHO
OTAY RANCHO
PLATE XXXI
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Drawn by R.H. Stretch, June 1888

of the irrigation supply of the reservoir is $4 000 000. The construction of the works has already added a value of $1 500 000 to the principal tract of five thousand acres which has been supplied with a complete system of water pipes, and another million to the value of town property in National City, and lands in its immediate vicinity.

The Plates accompanying this paper are:

Plate	XXIII.	Front View of Sweetwater Dam; Mount San Miguel in the distance.
"	XXIV.	Rear View of the Dam.
"	XXV.	View from lower end of Wasteway looking Northeast.
"	XXVI.	The Waste Weir.
"	XXVII.	The Tower, showing one of the Inlets with Basket Screen in position.
"	XXVIII.	Plan of Sweetwater Dam.
"	XXIX.	Elevation of Dam with details of Waste Weir.
"	XXX.	Section of Dam and Tower, with Outlet Conduit.
"	XXXI.	Map of San Diego Bay Region, showing location of the Sweetwater Dam and Reservoir and the District commanded by Pipe Lines from it.
"	XXXII.	View of Bear Valley Dam.
"	XXXIII.	Profile of Bear Valley Dam.

DISCUSSION.

JAMES B. FRANCIS, Past President Am. Soc. C. E.—I do not think I understand enough about this particular design to criticise it much. One thing strikes me as dangerous, and that is the wasteway being stopped up by planks or flash-boards and the relying on any one's opening it on proper occasion. Floods come very suddenly sometimes here, and I presume they do in that region. There would be great risk in this method.

H. W. BRINCKERHOFF, M. Am. Soc. C. E.—What is the surface area of the reservoir when full?

The SECRETARY.—721.86 acres.

Mr. FRANCIS.—What is the area of the water-shed?

The SECRETARY.—The water-shed, tributary to the dam, 186 square miles, of which one-third is above an elevation of 3 000 feet.

Mr. FRANCIS.—The area of the reservoir is then the one hundred and sixty-fifth part of the area of the water-shed, and a depth of 1 inch flowing off of the water-shed, corresponds to a depth on the reservoir of 165 inches. I take it, in that region, a rain-fall of 6 inches in twenty-four hours, or even more, should be provided for, which would require a very large spill-way.

The SECRETARY.—The paper refers to occasional rainfalls of great severity.

Mr. FRANCIS.—It seems that extraordinary provisions should be made for the overflow.

JAMES D. SCHUYLER, M. Am. Soc. C. E.—The dimensions of the wasteway were proportioned to carry a volume equal to or greater than the maximum flood discharge of the stream, as it was indicated by high water marks left by the freshet of 1884, the highest in the recollection of the oldest residents. With a blow-off gate equal to twenty per cent. of the capacity of the wasteway, which may be opened on the approach of a flood wave in advance of the rise of the water to the floor of the wasteway, the gate-keeper has full control of the floods. Moreover, before the parapet wall could be surmounted, the capacity of the wasteway would be double the flood discharge of the freshet of 1884, or about 3 000 cubic feet per second.

The flash-boards are not intended to be used until the end of the rainy season, to accumulate the later flow of the stream before it dwindles to its summer stage.

E. WEGMANN, Jr., M. Am. Soc. C. E.—In discussing the excellent paper which has just been read, the first point to which I wish to call attention, is that in this dam, as in all similar structures built recently in California, the plan of the wall is curved, being convex up-stream. While all engineers will readily admit that the curving of the plan adds

additional strength to a dam which has been built across a narrow valley, considerable difference of opinion exists as regards the advisability of adopting this plan of construction in the case of a wide valley. The pressures in the masonry resulting from a curved dam, acting as a horizontal arch, depend upon the radius of curvature and upon the thickness of the dam.

In considering the strains in the masonry, which may result from a dam acting as a horizontal arch, three cases may occur:

First.—The dam may act perfectly as an arch.

Second.—It may act imperfectly as an arch.

Third.—It may not act as an arch at all.

In the first case, the pressures in the masonry are to be calculated by the usual formulas.

In the second case, in which the amount of pressure transmitted by the dam to the lateral sides of the valley is supposed to be uncertain, we ought, in order to insure safety, to take an extreme assumption, namely, that the whole thrust of the water might have to be borne by the dam acting as an arch, which would make this case the same as the first.

Finally, the dam may not act as an arch at all, in which case the curving of the plan by adding to the length of the wall, would involve a waste of masonry.

That the stability of a dam may depend upon its acting as a horizontal arch, is proved by the Zola Dam in France, in which the line of pressure, reservoir full, falls considerably outside of the base of the wall. But in this case, the valley is narrow and the radius of curvature small, so that the maxima pressures arising from the arch action in the dam are within the limits of safety. Should we have, however, to deal with a wide valley, requiring a radius of about 1 000 feet for curving the plan of the dam, the pressures resulting from the arch action, as calculated by the usual formulas, would be found to be excessive. In all probability no part of the water-pressure is transmitted by a dam to the sides of a wide valley, as, owing to the length of the masonry, a considerable compression would have to occur before even a moderate amount of pressure could be transmitted.

Just where the distinction between a wide and a narrow valley should be made, must remain, in our present state of knowledge, a matter of judgment. A good rule to follow is, not to curve the plan of a dam, when the pressures arising in its masonry from its acting as a perfect arch are in excess of what is considered safe.

W. Barclay Parsons, M. Am. Soc. C. E.—What does Mr. Wegmann consider the limit of radius?

Mr. Wegmann.—I cannot give the exact radius, but a pressure of 14 tons per square foot should not be exceeded.

Mr. Parsons.—Acting as an arch?

Mr. WEGMANN.—Yes, sir; taking into consideration the dam acting as an arch.

Mr. PARSONS.—What radius is that?

Mr. WEGMANN.—I should think that it may be five or six hundred feet. The Zola Dam depends entirely upon its arched form for stability; it would be overturned were it not for its curved plan. This is the only case of this kind I recollect, unless the Bear Valley Dam, built in California recently, is another example. In this dam I think the line of pressure is very near the toe. The Bear Valley is narrow, and I think in that case there is no doubt but that the dam should be arched.

J. J. R. CROES, M. Am. Soc. C. E.—The pressure in pounds per unit of surface depends on the thickness of the dam at the elevation under consideration. The total arch pressure exerted at any point is equal to P times R; that is, the pressure per square foot on the face multiplied by the radius; this represents the total horizontal thrust on a voussoir joint, and this must be divided by the thickness of the dam in feet, to give the arch pressure per square foot of the joint.

Mr. SCHUYLER.—The Bear Valley Dam in San Bernardino County, Cal., cited by Mr. Wegmann, has no parallel among masonry dams in the civilized world. Its dimensions are so bold, it is so wide a departure from all previous theory and practice, and it has so long stood against the pressure of a full reservoir, that its profile is worthy of reproduction. See Plates XXXII and XXXIII.

At base it has a thickness of 22 feet, and it was carried up to the height of 16 feet the first year, and left in that condition during the winter. In the spring following, finding the cost of the original plan was to be greater than its projectors could afford, the structure was narrowed to a thickness of 8.5 feet, and carried to a height of 46 feet further, or 62 feet in all. At the top its thickness is but 2.5 feet.

These dimensions are more slender than have been heretofore reported, but I have taken pains to verify their correctness.

Considering the upper 46 feet of the dam independently, the resultant of gravity and water pressure (reservoir full) falls fully 15 feet outside the toe, or nearly twice the thickness of the masonry. Its radius is something over 250 feet, and the arch thrust on voussoirs is over 40 tons per square foot. The material is granite, laid in cut blocks, with Portland cement. The stability of the structure is, of course, entirely dependent on its arched form. The whole of the upper 46 feet acts as an arch, and withstands strains far beyond the theoretical limit. That it has stood for so long has been a matter of surprise to engineers who have known of its extraordinary frailness, but the fact that it has stood, with a full reservoir behind it (an enormous reservoir, too, covering some 2 000 acres, and impounding 10 000 million gallons), for at least three

years, should convey some useful suggestions on arch action in masonry dams worthy of careful study and investigation by the profession. It would seem to be a standing menace to the thousands of acres of vineyard and orange groves that lie in the valley below, and yet it remains a standing marvel, as curious as it is instructive. The company owning the dam are proposing to strengthen and increase the height of the structure.

Mr. WEGMANN.—Professor Rankine states that all calculations treating a dam as a horizontal arch are so indeterminate that they are of no value. He recommends, therefore, that a dam should be made sufficiently strong to resist by gravity alone, and that the plan be curved as an additional safeguard whenever the local conditions make this advisable. Of course, that leaves still a wide field for judgment. I should limit the curved plan to valleys of about 800 feet or less.

MR. CROES.—There are cases, where, in a valley of over 500 or 800 feet in width, a straight line would not be the most economical.

Mr. WEGMANN.—That would depend upon local conditions. The question of architectural effect may also deserve consideration; a curved dam may give a finer effect. The curved plan can do no harm under any circumstances, because when the valley is very wide the dam will simply not act as an arch; the only damage being a waste of masonry.

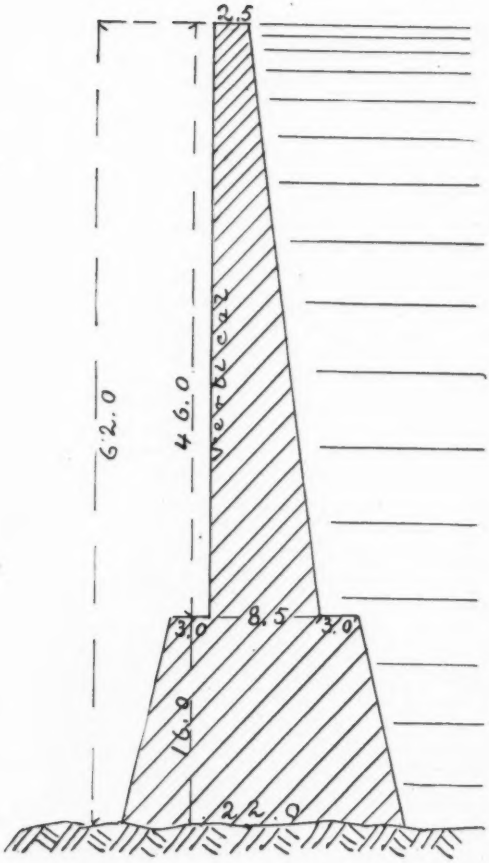
L. L. BUCK, M. Am. Soc. C. E.—Some curiosity is expressed to know why, if it was necessary to curve this dam at all, two different radii were adopted. The pressure against all points of the dam at equal depths below the surface of the water is equal, and hence it should ordinarily have a uniform curvature, if any, for the reason that, by compounding the curve, the flat portions will tend to thrust the sharp portions up-stream. I do not intend by my question to criticise the strength of the dam, but to ascertain the motives for compounding the curve.

Mr. SCHUYLER.—An examination of the plan will reveal the fact that on the up-stream face the curve is carried around with but one radius. It is only on the lower face next the waste-way that the compound curve was introduced to increase the thickness of the wall and relieve any possible shock that might be sustained by an extraordinary flow of water over the waste-way, as well as facilitate an increase of width of the latter. The compounding of the curve for a short distance simply adds a little thickness at one end of the dam in a curved wedge form, and does not affect the structure in any way except to increase its stability.

Mr. BUCK.—It may not be out of place here to say a few words regarding the subject of straight *versus* curved dams.

The dam is a retaining wall designed to hold the water of a river or other stream, to resist the pressure of this water and transmit such

Plate XXXII.—Bear Valley Dam.



PROFILE OF BEAR VALLEY DAM.—Plate XXXIII.

pressure to such immovable, natural foundation as the site selected will afford. It should be the aim of the engineer to effect this result with the least expenditure of capital that will render the structure perfectly safe. The nature of its service is such that it must retain its integrity of form, connection of parts to each other, and to the bottom and sides of the ravine or valley in which it is located.

Of the forms of masonry dams, there are two which have received the most attention in the discussions upon dams, viz.:

First.—A straight dam, with sufficient thickness to enable it, by its weight and strength, to safely resist the pressure brought against it by transmitting such pressure directly down to the natural rock on which the dam is built. In such a dam, the masonry on the down-stream side is compressed in excess of what pressure its own weight produces, while that produced on the up-stream side, caused by its own weight, is reduced by the pressure of the water. The effect of this action is to depress the lower side and elevate the upper. Consequently, the dam must lean down-stream to a greater or less extent, dependent upon the ratio of its height to its thickness. If the dam is properly designed and constructed, the amount of such leaning will be very slight. This is the most simple form of masonry dam.

Second.—This form of dam is curved in plan so as to present its convex side up-stream. The intention of this curved form is to cause the structure to act as an arch and transmit the pressure of the water to the sides of the valley or ravine. As the materials of which this arch is composed are shortened by such pressure, which means the shortening of the arc of the dam, while the ends must not approach each other, the curve will be flattened. Consequently this dam will also lean down stream. The amount of such leaning will increase in some ratio with the length and height of the dam, and the decrease of its curvature and thickness. It must have thickness enough to do its duty as an arch, to prevent the water finding its way through or underneath, and as the bottom must not move, it must be thick enough and heavy enough to prevent any part sliding on that below; or the whole sliding on the foundation. But if the thickness is too great, it may not allow the dam to lean enough to get the benefit of the arch form without cracking the masonry on the side exposed to the water. In this form of dam there can be no arch action at the bottom, nor, practically, for a considerable distance above, while its maximum is at the top. Hence it is practically a combination of the two systems and subject to the complicated stresses always attending such compound structures. As it must not have too flat a curvature, and as the chord of its arc must be somewhat less than twice its radius, it could, at best, be adapted only to situations where the valley is narrow in proportion to the height of the dam. In fact, its application should be limited to cases where the site is a very narrow, deep rock cañon or fissure, in which the distance from the middle of the

top of the dam to the solid rock at the sides of the cañon is not greatly in excess of that from the top to the solid rock at the bottom. Hence, remembering, 1st, that it is masonry that we are considering; 2d, that in the simplest structures of masonry, where durability is required, engineers consider it necessary to adopt a high factor of safety; and 3d, that the masonry structure is a dam requiring the most perfect integrity of construction in every part, it must be admitted that, when constructed on such a compound system as this last, involving so many uncertainties, it has no claims to being considered a scientific structure.

The idea of combining the two forms of dam, considered above, has gained some adherents among engineers. As I understand it, the plan proposed by its advocates consists of a curved dam. It is proposed, at the same time, to make it in every respect strong enough to be perfectly secure, had it not have been curved, but it is curved as a means of additional precaution. Here then we have a structure primarily built upon a system in which the pressure is transmitted to the solid rock on which it rests, possessing abundant stability, having a form of cross-section which cannot, and is not expected to, change (excepting to an almost inappreciable degree), and yet which it is proposed to render additionally secure by giving it a plan that could only accomplish its object by transmitting the pressure to very much more distant points, and could only do this after the first system had failed. No claim is made that the rock at the bottom does not afford every bit as reliable a foundation as that at the sides, or that it is not in a perfectly legitimate direction to transmit the pressure. And yet the advocates of this plan propose to increase the cost of the structure, by ten or fifteen per cent., by introducing such a complication as this. They find no fault with the simple and thoroughly scientific system of the gravity dam, in which the strains all lie in parallel direction, and end in the best possible foundation. Will some of them explain why, if additional security is required, it would not be best to seek it by simply extending the system on which the structure is designed?

Mr. WEGMANN.—The profile of the Sweetwater Dam has evidently been designed on scientific principles, and differs probably but slightly from a theoretical type having the minimum area. Among the many masonry dams which have been built within recent years in strict conformity to the requirements of theory, only one—the Habra Dam—has failed, and that was on account of bad workmanship and poor materials. The maxima pressures in the masonry, which in the first scientific profiles was limited to 4 to 6 tons per square foot, reached in recent structures 8 tons per square foot. In these dams the extreme positions of the lines of pressure, reservoir full or empty, are generally made to lie within the center third of the profile. I presume that this condition has been observed in the Sweetwater Dam.

The CHAIR.—I think Mr. Schuyler said they were inside the middle-third.

Mr. WEGMANN.—It would be interesting to ascertain that fact.

The SECRETARY (read from paper).—"In designing the plan of the higher structure," etc. (See page 206.)

Mr. WEGMANN.—The design seems to be bolder than I thought. As the line of pressure, reservoir full, falls considerably outside of the center-third of the profile, I think it would be hazardous to trust to the dam's strength to resist overturning merely by its weight. As the valley is narrow, however, at the site of the dam, the curved plan adds undoubtedly to the strength of the structure.

The weight of the masonry is given as 164 pounds per cubic foot. I know of only one dam in which the masonry was equally heavy, viz., Vyrnwy Dam near Liverpool. The stones of which the Sweetwater Dam was built must have had a very high specific gravity.

The SECRETARY.—The weight is given as 175 to 200 pounds per cubic foot.

Mr. WEGMANN.—That accounts for the great weight of the masonry.

The SECRETARY.—The stones placed in the wall are also said to have been large.

A. FTELEY, M. Am. Soc. C. E.—In connection with the question of weight of masonry, I will refer to two experiments made in 1885, to ascertain the probable weight of rubble stone masonry, such as might be used for the proposed construction of Quaker Bridge Dam.

Two blocks, about 1 cubic yard in volume, were built on a platform of rough stones, ranging in size from a common spall to about 3 cubic feet. The mortar used was made of Portland cement, mixed in the proportion of one of cement to two of sand. The stones used appeared to be gneiss.

After several months, the two blocks were measured and weighed with care, and found to have an almost identical weight of 156½ pounds per cubic foot. The computations subsequently made to determine the profile of the dam, were based on the weight thus ascertained.

Mr. CROES.—Would it be advisable to assume that a large mass of masonry would have the same specific gravity as small samples prepared in the manner indicated?

Mr. FTELEY.—The two samples just mentioned were taken from two different quarries in the vicinity of the proposed structure. The experimental blocks were built in the same manner as the mass of the dam should be built, and although it cannot be stated that the whole structure would be the same throughout, it seems fair (especially when it is admitted that the stones used in the final structure will be generally of a large bulk) to state that it is safe to use in the computations the weight determined experimentally.

Mr. FRANCIS.—One point occurs to me in connection with a curved dam. Suppose one to be built depending entirely on the curved form or arch; when the pressure of the water comes against it there would be a compression of the material and the arch would be flattened, which would cause it either to crack or slide on the bottom. I am taking an extreme case, that of a dam depending entirely on the arch form.

Mr. FTELEY.—Mr. Rankine, when speaking of the advisability of adopting a curved form for the plan of a masonry dam, says that "in the present state of science, the calculations of stability, treating the dam as a horizontal arch, are so uncertain as to be of doubtful utility." If, in connection with the above we consider a very long dam, the length of which is several times greater than the longest known arch, and built of rough stones without any attempt at any radial disposition of the joints, the superiority of the arch form does not appear a matter of certainty.

In such a case it is much more reasonable to suppose that whatever be the plan of the dam; straight, curved up-stream or down-stream, or with any irregular shape that may be determined by the topography of the ground, a failure would occur as it would in a retaining wall of indefinite length, between two weak points.

The uncertainty of relying upon a long curved dam as an arch, is so well recognized by those who advocate the curved form, that they recommend at the same time, as the most essential part of the design, a gravity section with a large margin of safety. Moreover, they admit that, in a curved dam of large span the structure will not act as an arch except after the force of gravity has been overcome. In such a contingency the condition of the structure designed to resist by its own gravity (say) twice the greatest expected strain, may well be imagined, and its chances to be then held by arch action would appear decidedly uncertain to those living below.

If the arch form, or any other form, is more economical than the straight line, it would be well to consider it favorably; if not, it is not advisable. Should it be advocated on account of architectural beauty, it remains with those in charge to determine whether the additional expenditure would be justified by the superior appearance of the structure.

C. B. COMSTOCK, M. Am. Soc. C. E.—Of course the strains at the bottom of the dam would be carried by the rock on which the dam rests, and as the lower course of the dam cannot move on this rock the arch effect there could not come into play at all; the arch effect only comes into play when there is some arch compression. When you strike the center of a vertical arch the crown sinks, the arch changing form. That cannot take place at the base of the dam. But at the top of the dam, although the water pressure is less than at the base, this pressure

acting as on the top of a vertical beam fixed at its base, bends the top of the dam down-stream. This bending compresses the horizontal arch formed by the top of the dam, and transmitted to the abutment, increases in some degree the power of the dam to resist overturn, considered only with a gravity section. You get some additional strength by giving it the curved form.

A gentleman suggests that there would be cracks in the dam. Of course, if the strains on the dam were nearly enough to break it by overturning there would be cracks on the upper face. I am supposing a dam so strong that those cracks would not take place, the bending of the dam not being sufficient to produce them. Then the arch resistance would increase the strength of the dam and reduce the factor of safety needed for a gravity section.

Mr. CROES. — Some experiments have recently been made by a board appointed by the Government, on the strength of brick arches, and the conclusion reached by the board was that these brick arches did not act as arches but as beams; that the masonry was monolithic, and that the strains were not arch strains at all, but were simply beam strains.

Mr. H. W. BRINCKERHOFF said that these experiments were referred to in the *Engineering and Building Record* of October 13th, 1888, and mention was there made of experiments noted in Vol. VIII., Van Nostrand's Engineering Magazine, October, 1872, as to the action as beams, of arches of brick.

THEODORE COOPER, M. Am. Soc. C. E.—I do not agree with the idea advanced that a masonry dam of large radius can exert much resistance as an arch. To do so it would be necessary for the arch to slide upon the foundations or at some intermediate points—an idea which no one would consider as possible or desirable. For a curved dam to act in any manner as an arch it must move from its original position. The bottom being fixed, motion can only take place by the bending of the cross-section down-stream, which means that the greatest arch action will be at the top and nothing at the bottom. This bending down-stream of each cross-section induces also crushing strains at the toe.

So that we must have, with our arch action, a resistance from the cross-section acting similarly to a pure gravity form. What is the relative value of the two actions? Arch action is dependent upon the amount of the deflection of the top of the dam down-stream. The crushing effect of the toe is measured by the same deflection. By assuming or calculating this deflection for a dam having a cross-section like the one before us, I am confident that the amount of arch action which could be found for a safe toe pressure, would be relatively very small, especially for long radii. The calculation is not a difficult one, and the least known

element of the calculation, the modulus of elasticity, entering into the deflection for both cases, can readily be eliminated.

After finding the theoretical amount of the arch resistance, we would still hesitate very much in assuming it to obtain to any such extent in actual practice owing to the absence of voussoirs and of homogeneity.

General COMSTOCK.—I did not propose to estimate the amount; I merely stated that I thought there was some resistance occurring in that way. I suppose if you were to take the top of that dam and press it with your hand you could bend the dam down-stream in some degree, greater or less, and the moment you bend the dam down-stream you bring into play the pressure of the arch, which adds a little to the strength of the dam against overturning. The moment you bend the dam down-stream you transfer a little pressure to the ends of the arch.

Mr. COOPER.—We agree technically, but where we differ is in the amount that would be gained. I think there would be a very small amount indeed. The dam is so large at the base in proportion to its height, that the deflection would be very minute.

General COMSTOCK.—The section of a dam 90 feet high, as I believe this was, would bend in some degree. The stone arch of the Chester Bridge of 200 feet span sank $2\frac{1}{2}$ inches at its crown at decentering, while for the Gloucester Bridge the sinking was 1 inch. As these sinkings gave severe compressions to the arch, it is probable that a sinking of $\frac{1}{10}$ of an inch would give a considerable compression in an arch not very flat.

For the purpose of estimate, suppose a foot in length of the dam to be replaced by a foot of granite in a single block 15 feet wide and 90 feet high; owing to its homogeneousness this block would have a much higher modulus than the actual dam, which, though thinner at top, is much thicker at its base. A horizontal down-stream pressure of a ton at the top of this stone would bend it down-stream by about $\frac{1}{30}$ of an inch, if W. Thomson's value of Young's modulus for stone be used, namely, 350×10^6 grammes per square centimeter. But if the value given in Hütte, 1883, be used (which seems much too small), a much larger deflection would result. As a ton's pressure would result from the top 9 feet of water, it seems possible that such a dam under the whole pressure might bend down-stream by $\frac{1}{10}$ of an inch.

Mr. SCHUYLER.—This estimate by General Comstock of the probable amount of bending of the dam down-stream with strains arising from a full reservoir, is quite reassuring. A movement of $\frac{1}{10}$ inch, or even twenty times that amount, might take place without appreciable results in the way of rupturing the masonry.

It seems clear to my mind that gravity and arch action must go hand in hand. When the reservoir is partly filled, in the case of the slender-

est of the pure arch type of dam, there is a point where the arch is not called into action, and resistance to pressure comes solely from the gravity of the structure. As the water rises the gravity point is passed, and then the excess of strain beyond the amount which the gravity of the mass is capable of resisting, is transmitted to the abutments. I do not think it can be maintained that when arch action begins, all the strains produced by water-pressure are necessarily borne by the arch, or any part of them, except those which the gravity of the dam cannot withstand. The dam must resist strain by its gravity to the extent of its weight, be that more or less, and from that point on it calls in the arch to its aid, and the two act together. You cannot get rid of the gravity of the mass in an arched dam. It is always there and always acting against an overturning force to the extent of its ability. I cannot conceive by what reasoning it should occur to any one that in order to permit a dam to act as an arch it should be necessary to increase the profile. If the profile is designed of such dimensions that it will resist by gravity the quiet pressure of water, the arch is not called into action until the pressure is increased by some extraordinary event, such as wave action, ice-thrust or earthquakes. This is the basis of reasoning upon which the profile of the Sweetwater Dam was designed. It was not designed to rely wholly upon the arch for its stability. If it had been, the experience of the Lola and Bear Valley dams would perhaps have justified a more slender profile. Nor was it intended to rely solely upon gravity for its factor of safety. It is stable under quiet pressure, and its arched form is believed to protect it from extraordinary shocks. The effect it was to produce upon the public mind in the inspiration of confidence was not disregarded, and in this respect its arched form adds such an appearance of solidity that it has been a marked success. It has been visited by thousands from all parts of the world, and no doubts are ever expressed of its thorough stability. It inspires the admiration, respect and confidence which every dam should inspire.

Mr. Cooper.—It has struck me, that the point which the author did not explain, is the building of three dams, one on top of the other; I do not think the author has anywhere given a description of the manner in which he put those three structures together.

Mr. Croes.—I think there is a deficiency in the paper in that respect. There was a large dam built before this was begun, and no description is given of the manner in which the masonry of the old dam is joined to the new one. If they were built in the same way, and care was taken, to make the joints between the old and the new work good, and the new work had time to set thoroughly before pressure was put upon it, I don't suppose it would make much difference.

Mr. Wegmann.—The dam was remarkably water-tight; is not that a proof that the joining must have been made thoroughly?

Mr. CROES.—The writer states that the dam is perfectly water-tight, but judging from some of the photographs of the dam here there is very evidently water coming through the dam at the point at which that joining was made.

The SECRETARY.—It is suggested that that may be water or it may be mortar.

Mr. CROES.—The mortar would not run out in that way unless there was water to bring it out.

A MEMBER.—The author did not say that the dam did not leak; the conduit did not leak.

Mr. CROES.—A few months ago I visited the Boyd's Corners Dam in Putnam County, N. Y., which is 60 feet high and was built about eighteen years ago, and there were in places streams of water running down the face of it. Immediately after this dam was built an earth embankment was built against the up-stream or water face. The joints in the masonry on this face of the dam were never thoroughly pointed before the earthen embankment was put against it. About 30 feet below the flow line there are places where the water is running down the face of the dam between the stones and the joints appear to be full of mortar, except at these few places. The face of this dam is of stones, from 2½ to 5 feet deep (bed), and from 18 to 20 inches rise, and the heart is of concrete, yet the water gets through it.

There was a small dam built of rubble masonry at New Rochelle, N. Y., about two years ago, which leaked very badly when the reservoir was first filled, the water passing through the masonry and spouting out with considerable force from some of the joints on the lower face. The proprietors had not considered it necessary to have the work inspected by an engineer during its construction, but called on one to remedy the defects which appeared on its completion. Mr. C. W. Hunt, M. Am. Soc. C. E., had charge of the repairs.

CHARLES W. HUNT, M. Am. Soc. C. E.—On examining the water-face it was found that there were a good many joints not properly filled with mortar. The water was then drawn down gradually, and masons working from a raft raked out the joints in the face and filled them up with cement mortar, well rammed in. Holes were found in which a stick could be pushed in three or four feet without meeting with any obstacle. In some holes as much as eight or ten pailsful of mortar were rammed in. The whole water face was pointed up in this way with well compacted mortar, and the dam does not leak at all now.

Mr. SCHUYLER.—The original structure of the Sweetwater Dam was encased by the secondary dam on both sides, as shown by the cross-section. It was narrow, and low, irregular in form and surface, with

numerous re-entering angles; and the union of the new masonry with the old was perfectly formed, with a grout of richer mortar than was commonly used. The bond was similarly formed between the secondary dam and its extension. The steps on the lower face of the former gave seat for the new work, and the omission of pointing mortar, as well as the irregularity of the face stones made the bond a good one. The work was allowed to set several months before it was subjected to much pressure.

As stated in the paper, the leakage through and under the secondary dam was entirely cut off by the new toe, but when the gates were closed and the reservoir allowed to fill, sweating took place through the masonry, principally above the level of the top of the secondary dam, and along the sides, through the minute seams of the bed-rock, carrying with it an efflorescence, forming an incrustation on the face which appears in the photographs. This sweating has dried up to a large extent. The total leakage through and under the wall at its maximum measured about 60 000 gallons daily. It is now reduced to about one-fifth that quantity, the greater portion of which is in minute trickling streams through crevices under the dam. All the leakage comes through without pressure, and can be stopped on the lower side, by pricking cotton or any fibrous substances into the tiny pinholes in the mortar through which it oozes. There is nothing like a jet or spurt of water anywhere apparent.